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NATIONAL DAM SAFETY PROGRAM. ORADELL RESERVOIR DAM (NJ00258), H--ETC(U)
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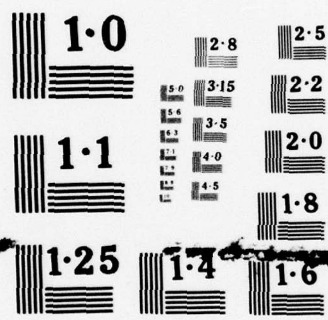
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LEVEL II

HACKENSACK RIVER BASIN

HACKENSACK RIVER, BERGEN COUNTY

NEW JERSEY

See 1473

ORADELL RESERVOIR DAM

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

NJ 00258

DEPARTMENT OF THE ARMY
PHILADELPHIA DISTRICT, CORPS OF ENGINEERS
CUSTOM HOUSE - 2D & CHESTNUT STREETS
PHILADELPHIA, PENNSYLVANIA 19106

DEPARTMENT OF THE ARMY
PHILADELPHIA DISTRICT, CORPS OF ENGINEERS
CUSTOM HOUSE—2 D & CHESTNUT STREETS
PHILADELPHIA, PENNSYLVANIA 19106

IN REPLY REFER TO
NAPEN-D

ACCESSION for		
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Honorable Brendan T. Byrne
Governor of New Jersey
Trenton, New Jersey 08621

22 SEP 1978

Dear Governor Byrne:

Inclosed is the Phase I Inspection Report for Oradell Reservoir Dam in Bergen County, New Jersey which has been prepared under authorization of the Dam Inspection Act, Public Law 92-367. A brief assessment of the dam's condition is given on the first three pages of the report.

Based on visual inspection, available records, calculations and past operational performance, Oradell Reservoir Dam, a high hazard potential structure, is judged to be in good overall condition. However, the spillway is considered inadequate since 37 percent of the Probable Maximum Flood (PMF) would overtop the dam. This is not viewed as a serious condition in that this dam is designed to withstand overtopping. To insure adequacy of the structure, the following actions, as a minimum, are recommended:

a. The adequacy of the spillway should be determined by a qualified professional consultant, engaged by the owner, using more sophisticated methods, procedures and studies within three months from the date of approval of this report. Any remedial measures necessary to insure the adequacy of the spillway should be initiated within calendar year 1979. In the interim, detailed emergency operation and evacuation plans and a warning system, should be promptly developed. Also, during periods of unusually heavy precipitation, around-the-clock surveillance should be provided.

b. Within six months from the date of approval of this report, engineering studies and analyses should be performed to assess the stability and erodability of the abutment and adjacent areas of the dam. Any remedial measures found necessary should be initiated within calendar year 1979. In addition, the seepage through the small hole in the right abutment retaining wall should be investigated in more detail.

c. Within three months from the date of approval of this report, the following actions should be taken.

(1) Repair sluice gates 1 and 2.

(2) Repair the areas of rip-rap failure and bank erosion on both sides of the downstream channel from the dam to the Oradell Avenue Bridge.

d. Within six months from the date of approval of this report, the following actions should be taken.

(1) The sluice gate outlet slab is spalled at many places and should be repaired.

(2) The small hairline crack that extends across the crest of the concrete spillway should be observed periodically to insure that this condition does not worsen.

(3) The one square foot spalled area on the front face of the dam should be patched.

(4) The expansion joints at both ends of the spillway should be resealed.

(5) Maintain observation of the moisture that is present on the front face of the dam at the "V"-notches and the longitudinal construction joints to determine whether this condition worsens.

(6) All expansion joints on the left and right retaining walls should be cleaned out and repaired, and the sealant should be replaced.

(7) Initiate a system of periodic inspections of the dam.

A copy of the report is being furnished to Mr. Dirk C. Hofman, New Jersey Department of Environmental Protection, the designated State Office contact for this program. Within five days of the date of this letter, a copy will also be sent to Congressman Andrew Maguire of the Seventh District and Congressman Harold Hollenbeck of the Ninth District. Under the provisions of the Freedom of Information Act, the inspection report will be subject to release by this office, upon request, five days after the date of this letter.

Additional copies of this report may be obtained from the National Technical Information Services (NTIS), Springfield, Virginia, 22161 at a reasonable cost. Please allow four to six weeks from the date of this letter for NTIS to have copies of the report available.

NAPEN-D
Honorable Brendan T. Byrne

An important aspect of the Dam Safety Program will be the implementation of the recommendations made as a result of the inspection. We accordingly request that we be advised of proposed actions taken by the State to implement our recommendations.

Sincerely yours,

Joel T. Callahan

1 Incl
As stated

JOEL T. CALLAHAN
Lieutentant Colonel, Corps of Engineers
Acting District Engineer

Cy furn:
Mr. Dirk C. Hofman, P.E., Deputy Director
Division of Water Resources
N. J. Dept. of Environmental Protection
P.O. Box 2809
Trenton, NJ 08625

ORADELL RESERVOIR DAM (NJ00258)

CORPS OF ENGINEERS ASSESSMENT OF GENERAL CONDITIONS

This dam was inspected on 12 June 1978 by Michael Baker, Jr., Inc. Consulting Engineers under contract to the U. S. Army Engineer District, Philadelphia, in accordance with the National Dam Inspection Act, Public Law 92-367

The Oradell Reservoir Dam, a high hazard potential structure, is judged to be in good overall condition. However, the spillway is considered inadequate since 37 percent of the Probable Maximum Flood (PMF) would overtop the dam. This is not viewed as a serious condition in that this dam is designed to withstand overtopping. To insure adequacy of the structure, the following actions, as a minimum, are recommended:

a. The adequacy of the spillway should be determined by a qualified professional consultant, engaged by the owner, using more sophisticated methods, procedures and studies within three months from the date of approval of this report. Any remedial measures necessary to insure the adequacy of the spillway should be initiated within calendar year 1979. In the interim, detailed emergency operation and evacuation plans and a warning system, should be promptly developed. Also, during periods of unusually heavy precipitation, around-the-clock surveillance should be provided.

b. Within six months from the date of approval of this report, engineering studies and analyses should be performed to assess the stability and erodability of the abutment and adjacent areas of the dam. Any remedial measures found necessary should be initiated within calendar year 1979. In addition, the seepage through the small hole in the right abutment retaining wall should be investigated in more detail.

c. Within three months from the date of approval of this report, the following actions should be taken.

(1) Repair sluice gates 1 and 2

(2) Repair the areas of rip-rap failure and bank erosion on both sides of the downstream channel from the dam to the Oradell Avenue Bridge.

d. Within six months from the date of approval of this report, the following actions should be taken.

(1) The sluice gate outlet slab is spalled at many places and should be repaired.

(2) The small hairline crack that extends across the crest of the concrete spillway should be observed periodically to insure that this condition does not worsen.

(3) The one square foot spalled area on the front face of the dam should be patched.

(4) The expansion joints at both ends of the spillway should be resealed.

(5) Maintain observation of the moisture that is present on the front face of the dam at the "V"-notches and the longitudinal construction joints to determine whether this condition worsens.

(6) All expansion joints on the left and right retaining walls should be cleaned out and repaired, and the sealant should be replaced.

(7) Initiate a system of periodic inspections of the dam.

APPROVED: Joel Callahan
JOEL T. CALLAHAN
Lieutenant Colonel, Corps of Engineers
Acting District Engineer

DATE: 22 September 1978

PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam - Oradell Dam, Bergen County, New Jersey
River - Hackensack River
Date of Inspection - 12 June 1978

ASSESSMENT OF GENERAL CONDITIONS

Oradell Dam is a unique structure consisting of a 331 feet long overflow weir section and a 68 feet long sluice gate section, both of which have maximum heights of approximately 25 feet. The weir section of hydraulically placed clay fill has a reinforced concrete envelope on its crest and downstream slope. This envelope is supported largely by steel braces and reinforced concrete counterforts which are in turn supported by a reinforced concrete floor with grade beams founded on timber piles. The dam is owned and operated by the Hackensack Water Company.

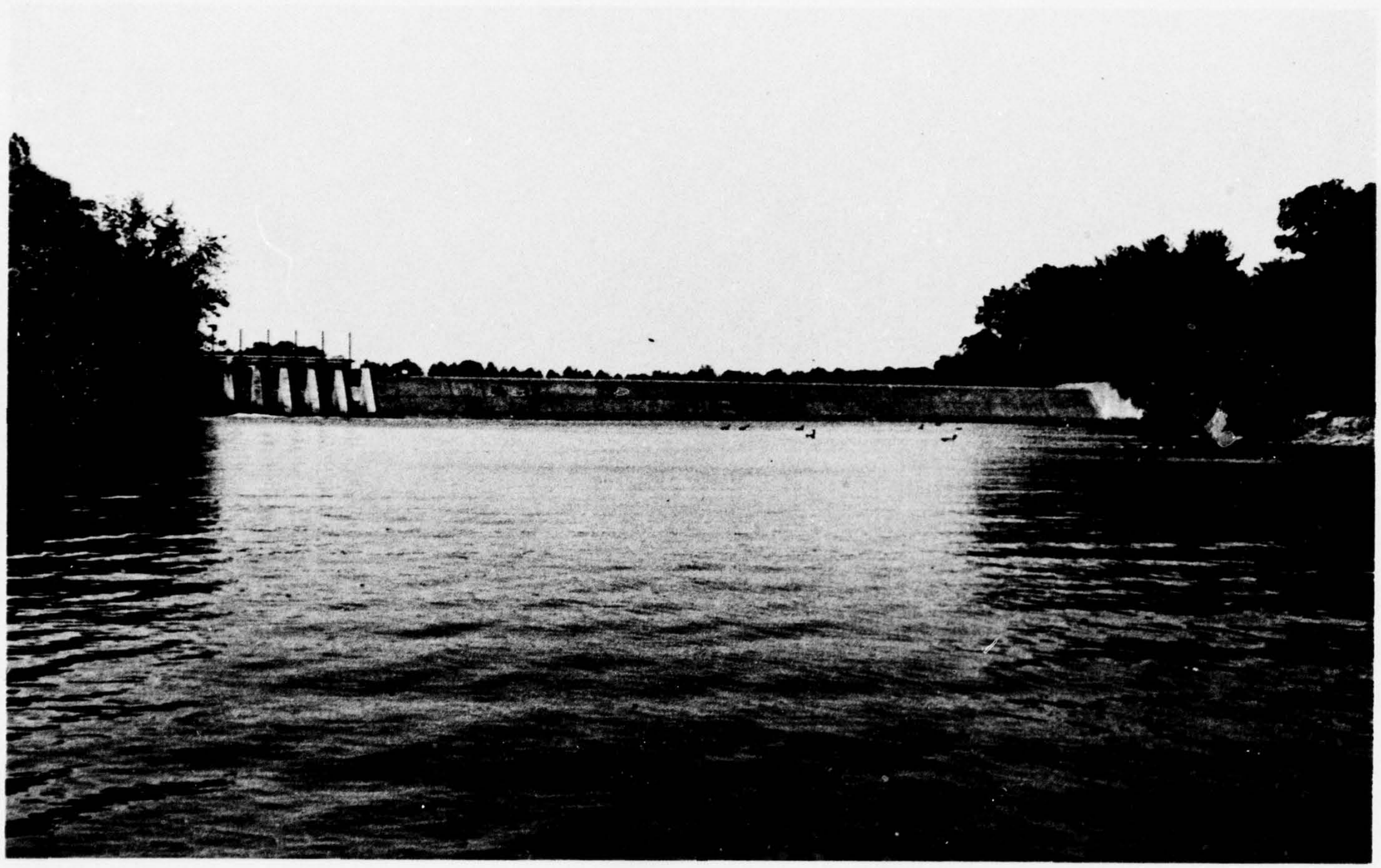
Visual inspections and review of engineering data in June and July 1978, indicate no serious deficiencies requiring emergency attention. The inspection showed the dam to be in good overall condition. It is recommended that the owner repair sluice gates 1 and 2 which were inoperable at the time of inspection, repair areas of riprap failure and bank erosion on both sides of the downstream channel from the dam to the Oradell Avenue Bridge, investigate the cause of seepage through the small hole in the right abutment retaining wall and take corrective action as necessary, repair areas of spalled or cracked concrete and expansion joints, develop emergency operations procedures for the dam and reservoir, and perform future periodic inspections of the dam.

Hydraulic/hydrologic analysis recently performed by a consulting engineering firm retained by the owner indicates that the spillway will not pass the Probable Maximum Flood without overtopping the dam. The analysis indicated that the dam and surrounding areas will be overtopped by approximately three feet of water. Therefore, the owner should initiate a detailed hydraulic and hydrologic study to more accurately assess the spillway capacity under Probable Maximum Flood conditions. After completion of the detailed hydraulic/hydrologic study, an evaluation of the stability and erodability of the abutment and adjacent areas of the dam should be performed.

MICHAEL BAKER, JR., INC.

Michael Baker, III, P.E.
Chairman of the Board and
Chief Executive Officer
Registration Number 13385

NAME OF DAM: ORADELL DAM



OVERALL VIEW OF DAM

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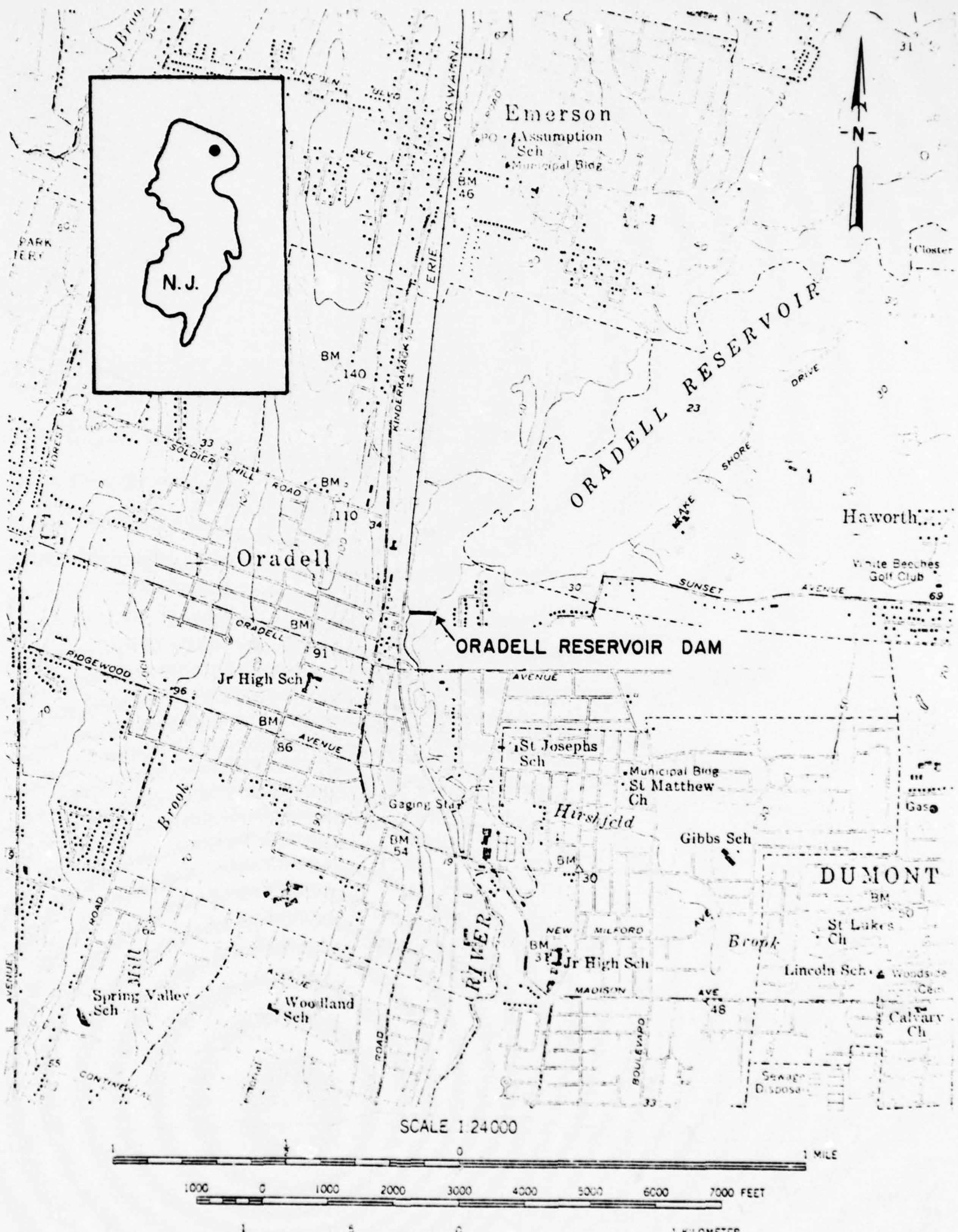


LOCATION PLAN
ORADELL RESERVOIR DAM

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
NAME OF DAM: ORADELL DAM, ID# NJ 00258

SECTION 1 - PROJECT INFORMATION

1.1 GENERAL

a. Authority - This report is authorized by the National Dam Inspection Act, Public Law 92-367, 92nd Congress, H.R. 15951 enacted 8 August 1972 and has been prepared in accordance with Contract No. DACW61-78-C-0141 between Michael Baker, Jr., Inc., and the U.S. Army Corps of Engineers, Philadelphia District.

b. Purpose of Inspection - The purpose of this inspection is to evaluate the general condition of Oradell Dam with respect to safety of the facility based upon available data and visual inspection.

1.2 DESCRIPTION OF PROJECT

a. Description of Dam and Appurtenances - Oradell Dam is a unique structure consisting of a 331 feet long overflow weir section separated from a 68 feet long sluice gate section by a three feet wide reinforced concrete training wall. The sluice gate section is located at the right (west) abutment and the weir section extends across the river channel to the left (east) abutment (see Plate 1). The dam and its appurtenances are described briefly in the following paragraphs. For a more detailed description, refer to Hill, N.S., Jr., "The Oradell Dam of the Hackensack Water Company," Transactions, American Society of Civil Engineers, Vol. 89, 1926, pp. 1181-1202.

The 20 feet high overflow weir section is of hydraulically placed clay fill with a reinforced concrete envelope on the upper portion of the upstream slope, the crest, and the entire downstream slope (see Plates 3 and 8). This envelope is supported largely by steel braces, reinforced concrete counterforts, and a reinforced concrete floor with grade beams beneath the counterforts (see Plate 3). The grade beams are supported by 14 inch diameter, 20 ton design capacity timber piles extending some 30 feet through glaciofluvial soils to underlying "hardpan," i.e., glacial till (see Plates 1, 3, 4 and 5). The downstream toe of the concrete envelope is thickened as a combination

toe block, pile cap, and bucket-type energy dissipator. This toe block contains horizontal two inch diameter drainpipes for relief of water pressures in the earth fill (see Plates 3 and 8). (The inclined drainpipes shown on Plates 3 and 8 at the downstream toe of the weir were apparently never installed.) A reinforced concrete apron extends 45 feet downstream from the weir toe block (see Plates 1, 3, 6 and 8).

The weir section, which was resurfaced in 1965, has crest El. 23.16 and toe El. 7.16 (see Plate 8) referenced to U.S.G.S. datum of Mean Sea Level (M.S.L.) which is 2.62 feet lower than the Mean High Tide (M.H.T.) datum used in the original design (see Plates 1 through 7). All elevations given in this report, except those on Plates 1 through 7, are referenced to M.S.L. datum.

The sluice gate section is of reinforced concrete with seven bays, each containing a seven by nine feet gate (see Plate 2). The sluice gate section is founded on concrete piers bearing on "hardpan," which is fairly shallow along the right abutment (see Plates 1 and 4). A reinforced concrete floor spans between the piers and a two feet wide concrete cutoff wall extends from the floor to "hardpan" (see Plate 2). The sluice gates were manually operated until 1959 when a portable electric gear drive unit suitable for operating one gate at a time was added. The operating platform at the top of the sluice gate section is at El. 25.6 and the gate exit inverts are at El. 2.6. A reinforced concrete apron extends approximately 50 feet downstream from the sluice gate section (see Plates 1 and 6).

A cutoff wall of steel sheet piling was driven to refusal on "hardpan." This sheet piling extends from the left (east) abutment concrete retaining wall along the spillway section to the training wall on the left (east) side of the approach channel to the sluice gates (see Plate 5). Tops of sheet piles are embedded in the concrete of the left abutment retaining wall, weir section floor slab (see Plate 3), and sluice gate approach channel training wall. The steel sheet piling forms a continuous cutoff with the concrete core wall of the sluice gate section (see Plate 2).

The upstream toe of hydraulically placed clay fill for the weir section was retained by an earth and timber crib dam which had been constructed in

1911. This crib dam with a crest at approximately El. 14.2 had a timber sheet pile cutoff extending about 20 feet below streambed to approximately El. -20. The crib dam had a width of about 10 feet and was curved in plan. The left (east) end, center and right (west) end of the crib dam were located about 45, 80 and 50 feet upstream, respectively, from the crest of the weir section. The right end of the crib dam was removed for the sluice gate approach channel and the training wall on the left side of the approach channel was tied into the crib dam to retain the hydraulic fill portion of the weir section.

Concrete gravity retaining walls on both ends of the dam were extended into the abutments as concrete core walls (see Plate 1). These retaining walls and core walls have top El. 25.6. The right abutment retaining wall and core wall are founded on "hardpan" (see Plate 1). The left abutment retaining wall and core wall are founded on 14 inch diameter timber piles, as is the sluice gate approach channel training wall (see Plate 5). As mentioned previously, the steel sheet pile foundation cutoff also extended into the left abutment and along the sluice gate approach channel training wall (see Plate 5). A reinforced concrete apron extends the full width of the sluice gate approach channel for a distance of about 90 feet upstream from the gates.

b. <u>Location</u> - Oradell Dam is located on the Hackensack River in the town of Oradell, Bergen County, New Jersey. The dam is located about one mile upstream from the town of New Milford and four miles upstream from the city of Hackensack.

c. <u>Size Classification</u> - The maximum height of the dam is 25 feet and the reservoir volume to spillway crest level is 10,026 acre-feet. Therefore, the dam is in the "Intermediate" size category as defined by the "Recommended Guidelines for Safety Inspection of Dams."

d. <u>Hazard Classification</u> - Low-lying portions of the towns of Dumont and New Milford along the reach of Hackensack River extending approximately one and one-half miles downstream from the dam contain an estimated 70 homes and 400 persons as well as the New Milford intake and treatment plant of the Hackensack Water Company and other facilities. In the event of failure of Oradell Dam, it is estimated that "more than a few" lives would be lost and the

economic losses would be "excessive." The dam is therefore classified in the "High" hazard category as defined by the "Recommended Guidelines for Safety Inspection of Dams."

e. Ownership - The dam is owned by Hackensack Water Company, 4100 Park Avenue, Weehawken, New Jersey 07087.

f. Purpose of Dam - The dam is used for water supply and limited recreation.

g. Design and Construction History - The existing facility was designed for the owner by Nicholas S. Hill, Consulting Engineer, New York, New York, over the period from 1911 to 1922. The dam was constructed from 1920 to 1922. Names and addresses of construction contractors are not readily available. The concrete weir envelope was resurfaced in 1965 (see Plate 8).

h. Normal Operational Procedures - Efforts are made to keep the reservoir full to the spillway weir crest, El. 23.16. Water is released through the sluice gates as necessary for intake and treatment downstream. Sluice gates are typically adjusted several times per day to balance discharge with downstream demand. The gates are never fully opened in normal operations. A gate operator visits and inspects the dam every two hours under normal conditions and every hour when rainfall exceeds 0.5 inch.

1.3 PERTINENT DATA

a. Drainage Area - The drainage area of Oradell Dam is 112 square miles.

b. Discharge at Damsite - The maximum known flow at the damsite through the gates and over the spillway is 3120 m.g.d. This flow occurred on 9 November 1977 with a head of approximately 1.6 feet on the spillway and the gates were not completely open.

c. Elevation (feet above M.S.L.) -

Top of Dam - 1) 25.6 - Sluice gate operating platform
2) 23.16 - Spillway weir crest
Maximum Pool (Design Discharge) - 23.6 (7970 c.f.s.)
(original design -- seven gates fully open and one foot water over spillway)
Normal Pool - 23.16
Streambed at Centerline of Dam - 0.8
Maximum Design Tailwater - 14.9

d. Reservoir (miles) -

Length of Normal Pool - 4.2 (Pool El. 23.16)

e. Storage -

At Spillway Crest (El. 23.16) - 10,026 acre-feet
Top of Dam - Not applicable

f. Reservoir Surface (acres) -

Top of Dam - 623
Spillway Crest - 623
Normal Pool - 623

g. Dam -

Type - Hydraulically placed earthfill with concrete envelope (ogee weir)
Length - 402 feet
Height - 25 feet
Top Width - 11.5 feet (sluice gate operating platform)
Side Slopes - Upstream - Variable (maximum three horizontal to one vertical [3:1])
Downstream - 0.87:1
Impervious Core - Hydraulically placed clay fill with downstream envelope of reinforced concrete
Cutoff - Steel sheet piling at upstream edge of pile foundation for weir section, wood sheet piling for original earth and timber crib dam (cofferdam at upstream toe of hydraulic earthfill weir section), concrete cutoff for sluice gate section.

h. Diversion and Regulating Tunnel - None

i. Spillway -

Type - Ogee weir (reinforced concrete envelope over earth embankment)
Length of Weir - 331 feet
Crest Elevation - 23.16 feet (M.S.L.)
Gates - None
Downstream Channel - Concrete apron extends 45 feet downstream from spillway toe in Hackensack River channel.

j. Regulating Outlets - Seven electrically operated sluice gates, each seven by nine feet in size.

SECTION 2 - ENGINEERING DATA

2.1 DESIGN

Design data reviewed included the following:

1) Reference Drawings:

   a) "Proposed Oradell Dam for Hackensack Water Co.," prepared by N.S. Hill, Jr., Consulting Engineer, New York, New York, 1911-1923 (numerous sheets, some of which are included as Plates 1 through 7 of this report)

   b) "Modifications to Oradell Dam - Plan, Section and Details," Drawing No. 400.1-16 prepared by Buck, Seifert and Jost, Consulting Engineers, Englewood Cliffs, New Jersey, January 1965 (included as Plate 8 of this report)

2) Hill, N.S., Jr., "The Oradell Dam of the Hackensack Water Company," Transactions, American Society of Civil Engineers (A.S.C.E.), Vol. 89, 1926, pp. 1181-1202; Discussions, pp. 1203-1212. (Plates 1, 2, 3 and 4 of this report are Figures 2, 3, 6 and 1, respectively, of the A.S.C.E. paper.)

3) Correspondence, memoranda, reports, photographs, calculations, and other documents in the microfiche files of the New Jersey Department of Environmental Protection (N.J.D.E.P.). Of particular importance are:

   a) "Report on Oradell Dam," by New Jersey Department of Conservation and Development Board of Consultants (W.H. Burr, H.T. Critchlow, and A.H. Pratt) 1 December 1921, 6 pp. plus appendices.

   b) "Annual Report - Oradell Dam," by G.M. Haskew, Jr., Hackensack Water Company, 22 April 1968, 2 pp.

4) Information furnished by J.J. Cannizzo, Director of Engineering Design and Construction, Hackensack Water Company, with his letters of 9 and 23 June 1978 to C.Y. Chen, Michael Baker, Jr., Inc.

5) Information furnished by representatives of the Hackensack Water Company in interviews during the field inspection on 12 June 1978 and in subsequent telephone conversations.

2.2 CONSTRUCTION

Site work for Oradell Dam began in 1911 and the dam was constructed from 1920 to 1922. The only readily available source of construction information is Hill's 1926 A.S.C.E. paper referenced above. Microfiche files of the N.J.D.E.P. contain limited additional construction information. The concrete weir section was resurfaced in 1965 (see Plate 8).

2.3 OPERATION

Information on the operation of Oradell Dam was obtained from the Hackensack Water Company during interviews at the time of inspection and in related correspondence and telephone conversations as noted in paragraph 2.1 above. This operational information was presented previously in paragraph 1.2.h.

2.4 EVALUATION

Engineering data obtained and reviewed are considered sufficient for purposes of this Phase I Inspection Report.

SECTION 3 - VISUAL INSPECTION

3.1 FINDINGS

a. General - The dam and its appurtenant structures were found to be in good overall condition at the time of inspection. The problems noted during the visual inspection are considered minor and do not require immediate remedial treatment. Noteworthy deficiencies observed are described briefly in the following paragraphs. The complete visual inspection check list is given in Appendix A.

b. Dam - The dam itself is in good condition. It was brought to the attention of Michael Baker, Jr., Inc. that the downstream face of the dam had grout pumped into it in 1965 and was then covered with an eight inch reinforced concrete surfacing.

There is a hairline crack at the front face and crest interface of the weir. This crack extends for the entire length of the spillway section. The crack is superficial in nature and is, in all likelihood, due to tensile stress along this interface.

A spalled area approximately one square foot in area was observed at the midpoint of the dam three feet below the weir crest on the downstream side.

At the expansion joints at either end of the spillway, the neoprene sealant has deteriorated completely and is no longer functioning. Seepage is present at the right expansion joint.

There is a small amount of seepage from longitudinal construction joints as is evidenced by calcite stains on the concrete adjacent to the joints. No moisture was observed at the time of inspection.

There is moisture present in the vertical "V"-notches on the downstream face of the concrete spillway. This condition may be due to a small amount of overflow or due to seepage. It is recommended that this condition be continuously observed to determine the source of the moisture and to insure against a further deteriorated condition.

c. Appurtenant Structures - Gates 1 and 2 were inoperable at the time of inspection. (Note: Gates are numbered consecutively from the right to the left side of the sluice gate section.) The Hackensack

Water Company is planning to repair these gates as soon as practicable.

The downstream portion of the left abutment retaining wall has two vertical expansion joints that have spalled badly and the sealant material has also deteriorated. At one time, the joints had been regrouted but the grout has since deteriorated. The downstream end of the wall has spalled badly for two feet in length, three to four feet above the waterline.

The downstream portion of the right abutment retaining wall is in about the same condition as the left wall. There are spalled conditions approximately one foot wide adjacent to each of four expansion joints. The base of the wall is badly spalled at water level. This condition was in all likelihood brought about by the larger volume and higher velocity of water that passes through the sluice gates adjacent to the retaining wall. Downstream from the sluice gates, there is a one-fourth to one-half inch hole at approximately the midpoint of the wall that has a constant flow of water (estimated one to two G.P.M.) spouting out. This condition may be due to normal groundwater flow from higher ground along the right bank or it may be due to seepage through the right abutment. This condition had been present for some time as there is evidence of spalling below the spout. The source of this water flow should be investigated. As far as corrective measures are concerned, if the source is such that no major problem is raised, the hole may be left to serve as a weep hole.

The gate walls downstream of and adjacent to the sluice gates are in fair condition with some spalling at the toe due to high outlet velocities from the sluice gates. This condition is confined mainly to the gate walls adjacent to sluice gates 3 and 4 which are used more frequently than the remaining gates.

The headwalls above gates 3, 4, 5 and 7 are slightly spalled over an area seven feet long by two feet wide.

The sluice gate outlet slab is in fairly bad condition. It is spalled badly at many places which Michael Baker, Jr., Inc. has been informed is due to ice action.

d. Reservoir Area - No problems observed.

e. Downstream Channel - Minor sedimentation and aquatic vegetation extend along the left bank in a slack water area extending approximately 100 feet downstream from the dam.

Localized areas of riprap failure and bank erosion occur along the left bank from the dam to the "point" about 400 feet downstream from the dam and around this "point" to the Oradell Avenue Bridge situated 800 feet downstream from the dam. Other localized areas of riprap failure and bank erosion occur along the right bank from the dam to the Oradell Avenue Bridge.

The main hydraulic constraint in the downstream channel is the Oradell Avenue Bridge. The bridge has a clear opening approximately 90.0 feet wide by 5.5 feet high and there is evidence of scour on both sides of the channel approaching the bridge.

3.2 EVALUATION

a. Dam - The small hairline crack that extends across the crest of the concrete spillway should be observed periodically to insure that this condition does not worsen.

The one square foot spalled area on the front face of the dam should be patched.

The expansion joints at both ends of the spillway should be resealed.

Maintain observation of the moisture that is present on the front face of the dam at the "V"-notches and the longitudinal construction joints to determine whether this condition worsens.

b. Appurtenance Structures - All expansion joints on the left and right retaining walls should be cleaned out and repaired, and the sealant should be replaced.

Patch all spalled areas. (Note: All concrete repair work is strictly cosmetic in nature. There is no evidence of any concrete that has deteriorated to the point that structural integrity is affected.)

The seepage through the small hole in the right abutment retaining wall should be investigated in more detail.

Repair gates 1 and 2 as they were inoperable at the time of inspection.

c. Reservoir Area - No problems observed.

d. Downstream Channel - Sediment and vegetation in the slack water area immediately downstream from the left end of the weir section is inconsequential and will be scoured away during the next period of flood flow.

Localized areas of riprap failure and bank erosion along both banks from the dam to the Oradell Avenue Bridge present only minor environmental problems. These areas should be repaired during normal maintenance operations using riprap stone of appropriate sizes and durability. Graded filters of granular soil may be necessary beneath the riprap in certain locations, e.g., left bank "point" 400 feet downstream from dam, to prevent fine glaciofluvial soils from washing out from under the riprap.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 PROCEDURES

General operational procedures are discussed in paragraph 1.2.h.

There is no formal written procedure for emergency operation of the dam and reservoir or for downstream evacuation in the event of impending catastrophe. As noted in paragraph 1.2.h., operations personnel visit the dam at two hour intervals under normal conditions and at one hour intervals when rainfall exceeds one-half inch. When necessary, operations personnel visit the dam more frequently or remain at the dam to adjust the sluice gates.

Water levels and discharges at dams upstream and downstream from Oradell Dam are carefully monitored in order to establish discharges from Oradell Reservoir. U.S. Geological Survey (U.S.G.S.) stream and rain gages located 0.6 mile downstream from Oradell Dam are also monitored in this regard. Operations personnel receive gate setting instructions by two-way radios in their vehicles. In addition to these radios, personnel also have good access to telephone communications in case of emergency.

It is recommended that a formal emergency procedure be prepared and prominently displayed and furnished to all operating personnel. This should include:

1) How to operate the dam during an emergency.

2) Who to notify, including public officials, in case evacuation from the downstream area is necessary.

3) Procedures for evaluating inflow during periods of emergency operation.

4) Methods of removal of flow regulating devices to allow rapid drawdown of the lake under emergency conditions.

In addition, the owner should assist public officials in developing an emergency evacuation plan for areas which will be inundated by a flood or affected in the event of a dam failure.

4.2 MAINTENANCE OF DAM

The dam is well maintained by the Hackensack Water Company. Formal, and informal inspections of the dam are made periodically. Maintenance is performed as required.

4.3 MAINTENANCE OF OPERATING FACILITIES

Refer to paragraph 4.2 above. The Hackensack Water Company is presently planning to repair sluice gates 1 and 2, which were inoperable at the time of inspection.

4.4 WARNING SYSTEMS

A regular inspection occurs every two hours and if rainfall exceeds one-half inch this inspection interval becomes hourly. Rain and streamflow gages within the watershed are observed, and this information is relayed to the central office. The information along with adjacent reservoir levels is evaluated and the appropriate discharge is released through the gates.

There are no special emergency procedures in the event of failure. However, the Civil Defense unit and government officials would be notified. An emergency warning procedure should be developed as recommended above in paragraph 4.1.

4.5 EVALUATION

The operating procedures are adequate for their intended purposes. Records are kept on a daily basis of gate operations and water levels in the reservoir. A formal emergency procedure should be developed, however, as noted in paragraph 4.1 above.

SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1 EVALUATION OF FEATURES

a. Design Data - Hydrologic and hydraulic design data in the microfiche files of the N.J.D.E.P. were reviewed along with data included in Hill's 1926 A.S.C.E. paper.

The July 1978 "Report on the Investigation of Safety of Oradell Dam" by Buck, Seifert, and Jost, Inc., Consulting Engineers was also reviewed and is attached in Appendix C.

b. Experience Data - Operational data furnished by the Hackensack Water Company were reviewed.

c. Visual Observations - No evidence of past flood damage to the dam or appurtenant structures was observed during the field inspection.

d. Overtopping Potential - The Oradell Dam is classified as a "High" hazard-"Intermediate" size dam requiring evaluation for a spillway design flood equal to the Probable Maximum Flood (P.M.F.). The spillway is a 331 feet long concrete ogee with a crest El. 23.16 feet M.S.L. Top of dam is El. 25.6 feet M.S.L. The spillway rating curve developed in the safety investigation report (Appendix C) showns the maximum discharge at top of dam El. 25.6 feet M.S.L. to be approximately 13,000 c.f.s.

The hydrologic analysis (Appendix C) of Oradell Dam was completed by the owner's consultant. The analysis indicates that the peak inflow discharge for the P.M.F. would be approximately 49,500 c.f.s. Using flood routing methods, the P.M.F. was routed through the reservoir and dam, and was found to overtop the dam by approximately 3.1 feet, cresting at El. 28.70 feet M.S.L. with a peak outflow of 35,750 c.f.s. The spillway, therefore, can only pass approximately 36 percent of the P.M.F. Consequently, according to the criteria specified in "Recommended Guidelines for Safety Inspections of Dams" and the analysis in Appendix C, the spillway is considered to be inadequate.

e. Emergency Drawdown of Lake - The outlet works of Oradell Reservoir consist of seven 7.0 feet by 9.0 feet sluice gates used to release water as required for the intake system of the Hackensack

Water Company, located approximately 0.6 mile downstream. The gates can be opened for emergency drawdown. According to the information available in the microfiche files of the N.J.D.E.P., at the spillway El. 23.16 M.S.L., the seven gates have a design capacity of approximately 6956 c.f.s. Neglecting inflow rate and variable head, the lake could be drained in less than one day with all seven gates open. It would take about one day under the above conditions with the five gates currently in operation.

SECTION 6 - STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations - No structural inadequacies were noted during the visual inspection of the dam.

b. Design and Construction Data - Readily available information on design and construction of Oradell Dam was reviewed. The most significant information sources were Hill's 1926 A.S.C.E. paper and the 1921 Board of Consultants report, both of which were referenced in paragraph 2.1 of this report. The Board of Consultants established by the New Jersey Department of Conservation and Development in 1921 consisted of three eminent dam engineers of that era: W.H. Burr, H.T. Critchlow, and A.H. Pratt. These engineers reviewed design information and construction progress through November 1921 and presented the following opinion on page 6 of their report of 1 December 1921:

> "With the provision for satisfactory operating conditions of the gates and the additional protection against scour at the downstream side of the dam, it is our judgement that the plans of the dam are in accordance with good engineering practice and the construction work appears to have been done in such a manner as to secure a safe and stable structure."

(Downstream scour protection recommended by the Board of Consultants was subsequently added; see Plates 1, 3, 6 and 7 of this report.)

It is obvious from the description in paragraph 1.2.a. of this report that Oradell Dam is a unique structure for which the stability requirements of the "Recommended Guidelines for Safety Inspection of Dams" are not applicable. Structural stability of Oradell Dam must be considered within the historical context of the period of its design and construction when dam building was more of an art than a science. In this regard, the above-quoted opinion from the 1921 Board of Consultants report must be accepted as verification of structural stability at the time of dam design and construction.

The structural stability of Oradell Dam has been evaluated by Buck, Siefert, and Jost, Inc., Consulting Engineers of Englewood Cliffs, New Jersey for the owner, the Hackensack Water Company. A report entitled "Investigation of Safety of Woodcliff Lake, Oradell, and Lake Tappan Dams of Hackensack Water Company and Lake De Forest Dam of Spring Valley Water Company," dated July 1978 was submitted to the Hackensack Water Company by Buck, Siefert, and Jost, Inc. Portions of this report pertaining to Oradell Dam was photocopied and forwarded in August 1978 to Michael Baker, Jr., Inc., by the Hackensack Water Company. These pertinent sections of the consultant's report are included in this Phase I Inspection Report as Appendix C. The consultant evaluated the structural stability of the spillway section and the sluice gate section using methods similar to those presented in EM 1110-2-2200, "Gravity Dam Design," by the U.S. Army Corps of Engineers. The loads considered in the analysis of the spillway section included the weight of the structure, water load under peak outflow conditions, tailwater, uplift, and earth pressure of the internal earth filling. Material properties used in the analysis included 150 p.c.f. for the unit weight of concrete; 100 p.c.f. dry unit weight, 63 p.c.f. submerged unit weight, and the angle of internal friction equal to 30 degrees for the earth filling. A similar loading was used for the sluice gate section stability analysis except the passive resistance of the earth filling adjacent to the concrete support piers was also considered. The consultant summarized the stability analyses in the text of the report as follows.

> "Stability of the spillway section is excellent as the resultant falls within the middle third of the base. Sliding resistance is ample without the shear resistance of the piling. Stresses are low and are positive, with the load on the foundation assumed to be carried entirely by the piers. Sliding resistance of the sluiceway is satisfactory when passive resistance of the backfill is taken into account. Stresses are low. A small amount of tension exists on the highest pier. However, this pier adjoins the spillway section which has piling. The resultant falls slightly in the downstream third of the base which indicates a small amount of tension but not overturning. The position of the resultant is more favorable as the piers reduce in height."

The analysis performed by the owner's consultant indicates the stability of the spillway and sluice gate sections are adequate. This conclusion is supported by the long history of successful behavior of the dam, the low hydraulic head on the dam, the fact that no structural inadequacies were noted during the visual inspection by Michael Baker, Jr., Inc., and the high level of operation and maintenance attention focused on the dam by the Hackensack Water Company. However, it must be indicated that the hydraulic/hydrologic analysis determined that the dam, abutments and surrounding areas will be flooded (overtopped) by approximately three feet of water under P.M.F. conditions. It is recommended, therefore, that the abutment and adjacent areas of the dam be evaluated for stability and erodability under the P.M.F. conditions.

c. Operating Records - Nothing in the readily available operating records suggests structural inadequacy of the dam.

d. Post-Construction Changes - The only post-construction change of structural significance was resurfacing of the concrete weir envelope in 1965 (see Plate 8). This resurfacing improved structural stability of the dam by an indeterminate amount.

e. Seismic Stability - Oradell Dam is located in Seismic Zone 1 according to the "Seismic Zone Map of the Contiguous United States" Figure 1, p. D-30, "Recommended Guidelines for Safety Inspection of Dams." This is a zone of low seismic activity. Experience indicates that dams in Seismic Zone 1 will have adequate stability under seismic loading conditions if they have adequate stability under static loading conditions. As indicated in paragraph 6.1.b. above, Oradell Dam is considered to have adequate static stability, so further consideration of seismic stability is not warranted for this Phase I Inspection Report.

SECTION 7 - ASSESSMENT, RECOMMENDATIONS/REMEDIAL MEASURES

7.1 DAM ASSESSMENT

a. Safety - There are no findings, as a result of this Phase I Inspection, from which a detrimental assessment of the structural stability of the dam can be rendered. The spillway capacity was analyzed by a consulting engineering firm retained by the owner. This analysis and the structural stability assessment by the same consulting firm has been presented as Appendix C of this report. The results of this analysis indicate that the dam will be overtopped by approximately three feet of water under P.M.F. conditions.

b. Adequacy of Information - The information available and the visual observations made during the field inspection are considered sufficient for this Phase I Inspection Report.

c. Urgency - Repairs and remedial work recommended in Section 3 and 4, and paragraph 7.2, below, should be accomplished soon to prevent further deterioration.

d. Necessity for Further Investigation - The visual inspection of the dam and review of information available has indicated the need for additional investigation of certain items. These are:

1) A further detailed hydraulic and hydrologic study should be completed to more accurately assess the spillway capacity under P.M.F. conditions.

2) After completion of the detailed hydraulic/hydrologic study, an assessment of the stability and erodability of the abutment and adjacent areas of the dam should be performed.

3) The seepage through the small hole in the right abutment retaining wall should be investigated in more detail.

7.2 RECOMMENDATIONS/REMEDIAL MEASURES

The inspection revealed certain items of rehabilitation or other work which should be accomplished very soon by the owner. These are:

1) Repair of sluice gates 1 and 2 which were inoperable at the time of inspection. (We understand this repair work will be done as soon as practicable.

2) A further detailed hydraulic and hydrologic study should be completed to more accurately assess the spillway capacity under P.M.F. conditions.

3) After completion of the detailed hydraulic/ hydrologic study, an assessment of the stability and erodability of the abutment and adjacent areas of the dam should be performed

4) Repair of areas of riprap failure and bank erosion on both sides of the downstream channel from the dam to the Oradell Avenue Bridge.

5) The seepage through the small hole in the right abutment retaining wall should be investigated in more detail. However, this seep hole should remain open to allow relief of the water pressure until the cause of the seepage is identified and remedial measures implemented.

6) Develop emergency operations procedures for the dam and reservoir. This should include:

   a) How to operate the dam during an emergency.

   b) Who to notify, including public officials, in case evacuation from the downstream area is necessary.

   c) Procedures for evaluating inflow during periods of emergency operation.

   d) Methods of removal of flow regulating devices to allow rapid drawdown of the lake under emergency conditions.

In addition, the owner should assist public officials in developing an emergency evacuation plan for areas which will be inundated by a flood or affected in the event of a dam failure.

The inspection also disclosed several other items which should be accomplished soon by the owner.

1) The sluice gate outlet slab is spalled at many places and should be repaired.

2) The small hairline crack that extends across the crest of the concrete spillway should be observed periodically to insure that this conditions does not worsen.

3) The one square foot spalled area on the front face of the dam should be patched.

4) The expansion joints at both ends of the spillway should be resealed.

5) Maintain observation of the moisture that is present on the front face of the dam at the "V"-notches and the longitudinal construction joints to determine whether this condition worsens.

6) All expansion joints on the left and right retaining walls should be cleaned out and repaired, and the sealant should be replaced.

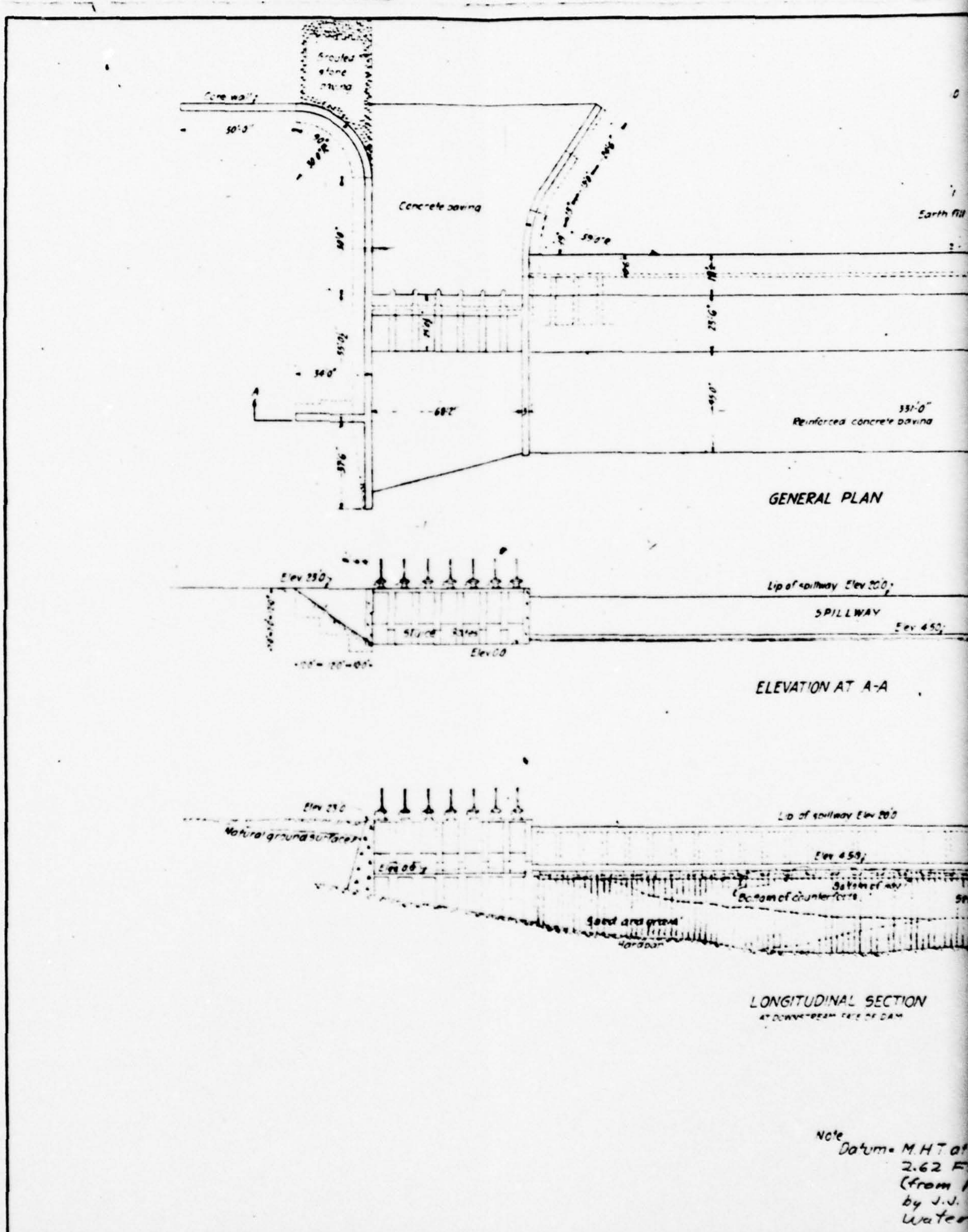
In the future, periodic inspections of the dam should be performed with particular attention directed to:

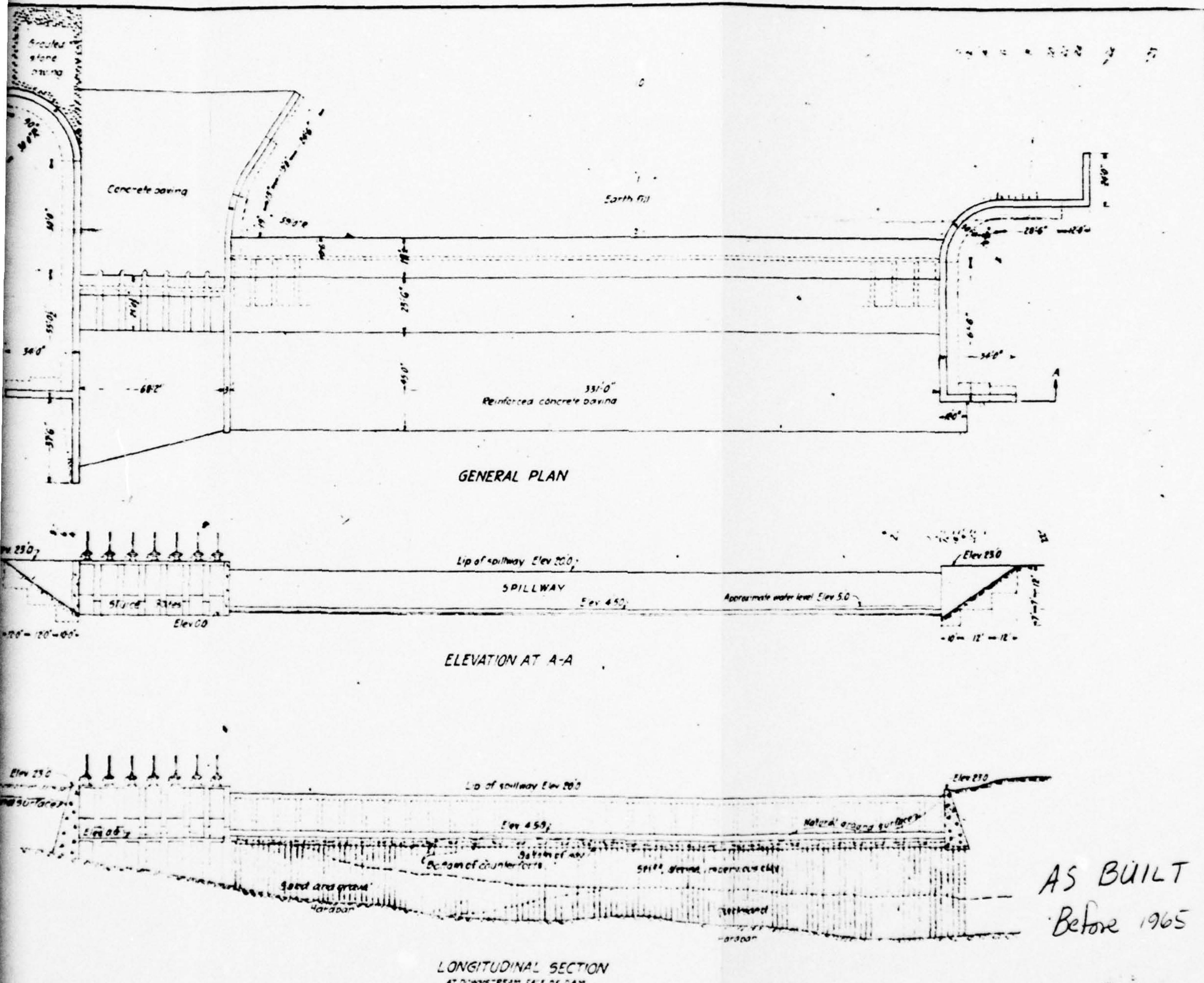
1) Sluice gate operational capability.

2) Concrete deterioration.

3) Riprap failure and bank erosion along the downstream channel.

PLATES


Grouted stone paving
Core wall
Concrete paving
Earth fill
Reinforced concrete paving
331'-0"
GENERAL PLAN
Lip of spillway Elev 20.0
SPILLWAY
Sluice gates
Elev 0.0
ELEVATION AT A-A
Elev 25.0
Lip of spillway Elev 20.0
Natural ground surface
Bottom of counterforts
Sand and gravel
Hardpan
LONGITUDINAL SECTION
AT DOWNSTREAM FACE OF DAM
Note
Datum - M.H.T
2.62
(from
by J.J.
Water



AS BUILT
Before 1965

FIG 2

GENERAL PLAN, ELEVATION AND SECTION
ORADELL DAM
HACKENSACK WATER COMPANY

Note
Datum = M.H.T. at Sandy Hook
2.62 Ft above USGS Datum MSL
(from letter of 23 June 1978
by J.J. Cannizzo, Hackensack
Water Co., to C.Y. Chen, MBurr)

Nicholas S. Hill, Jr.
Consulting Engineer
112 East 19th Street
New York City

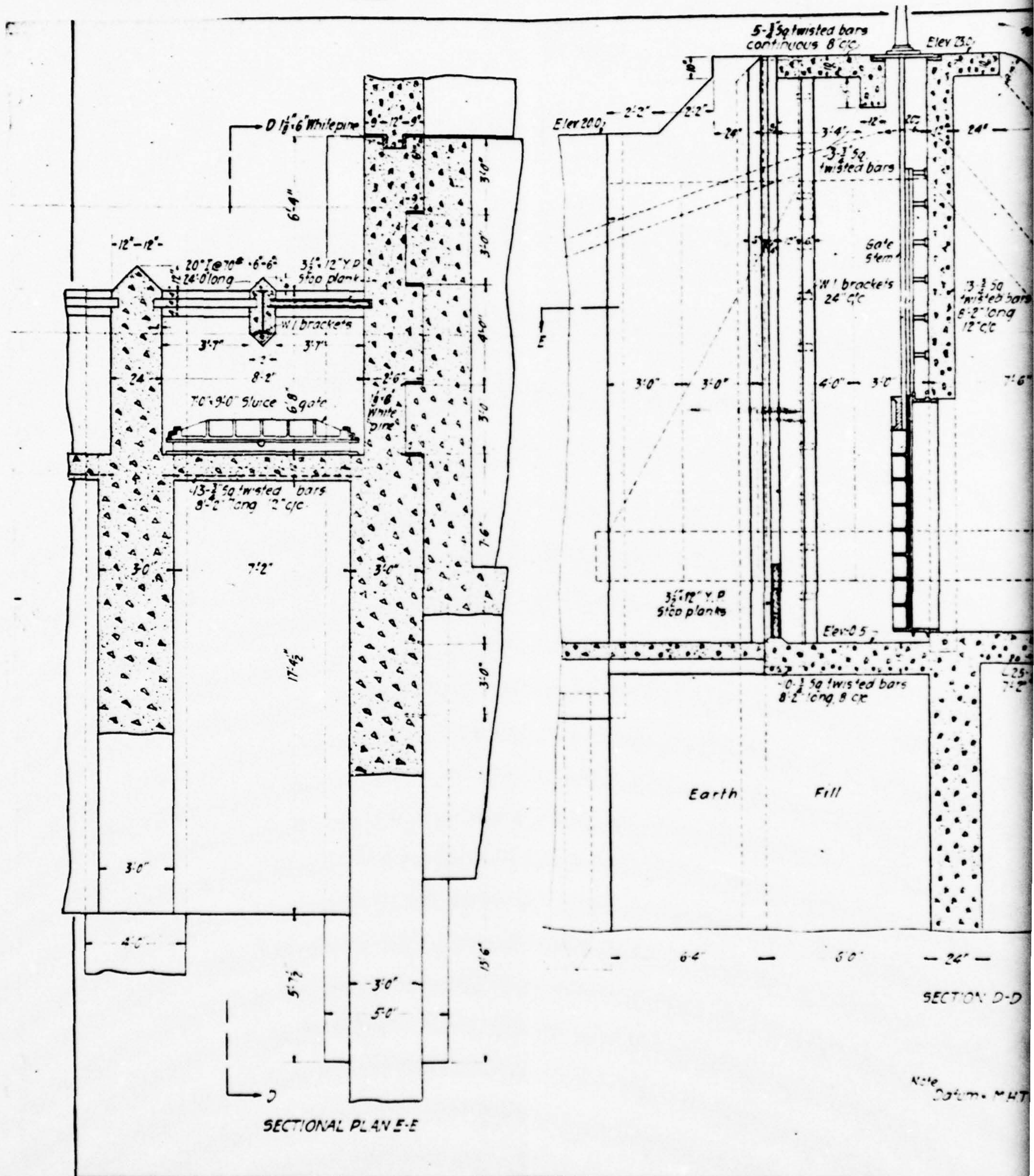
Scale 1:30

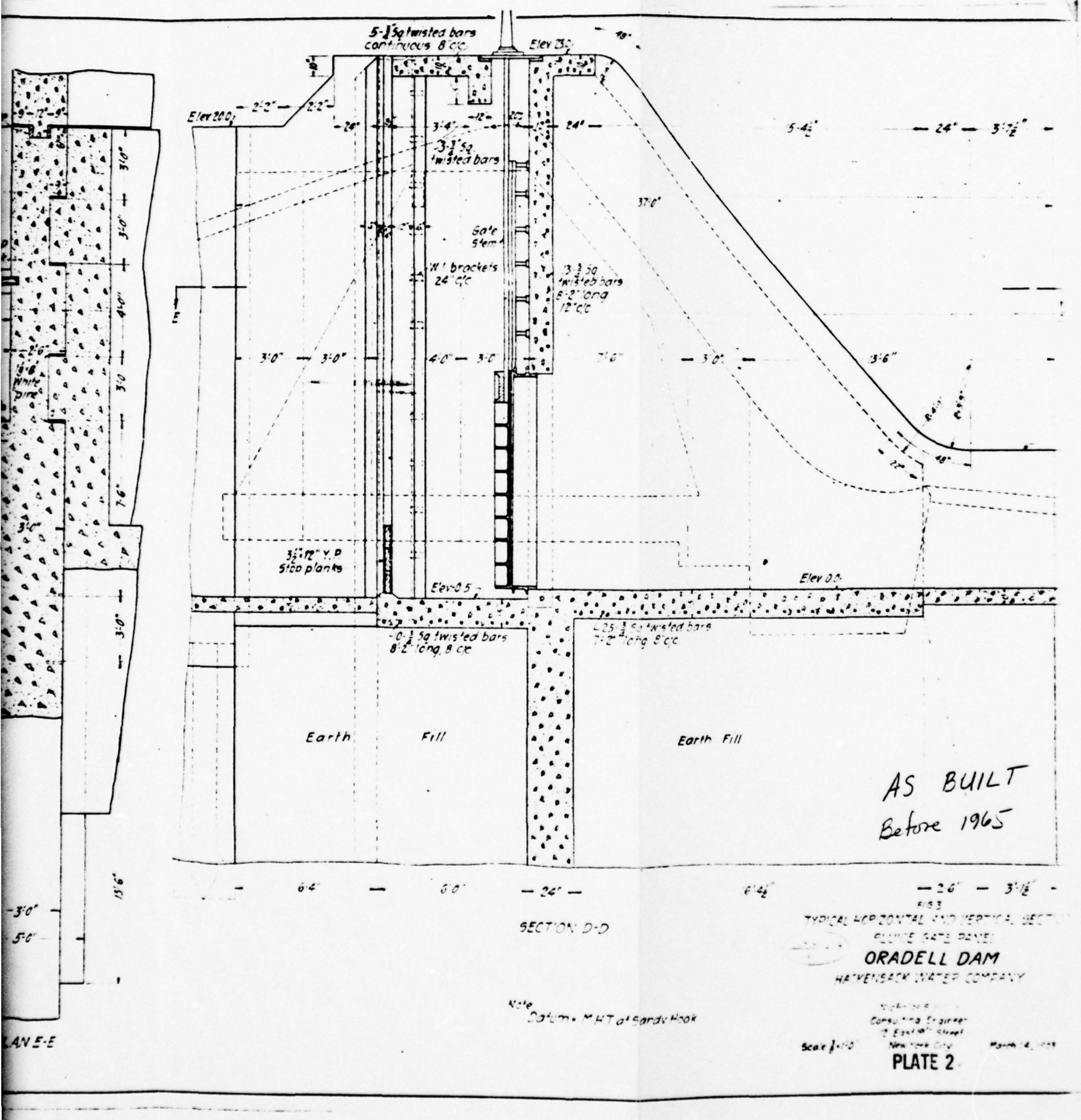
March 7, 192?

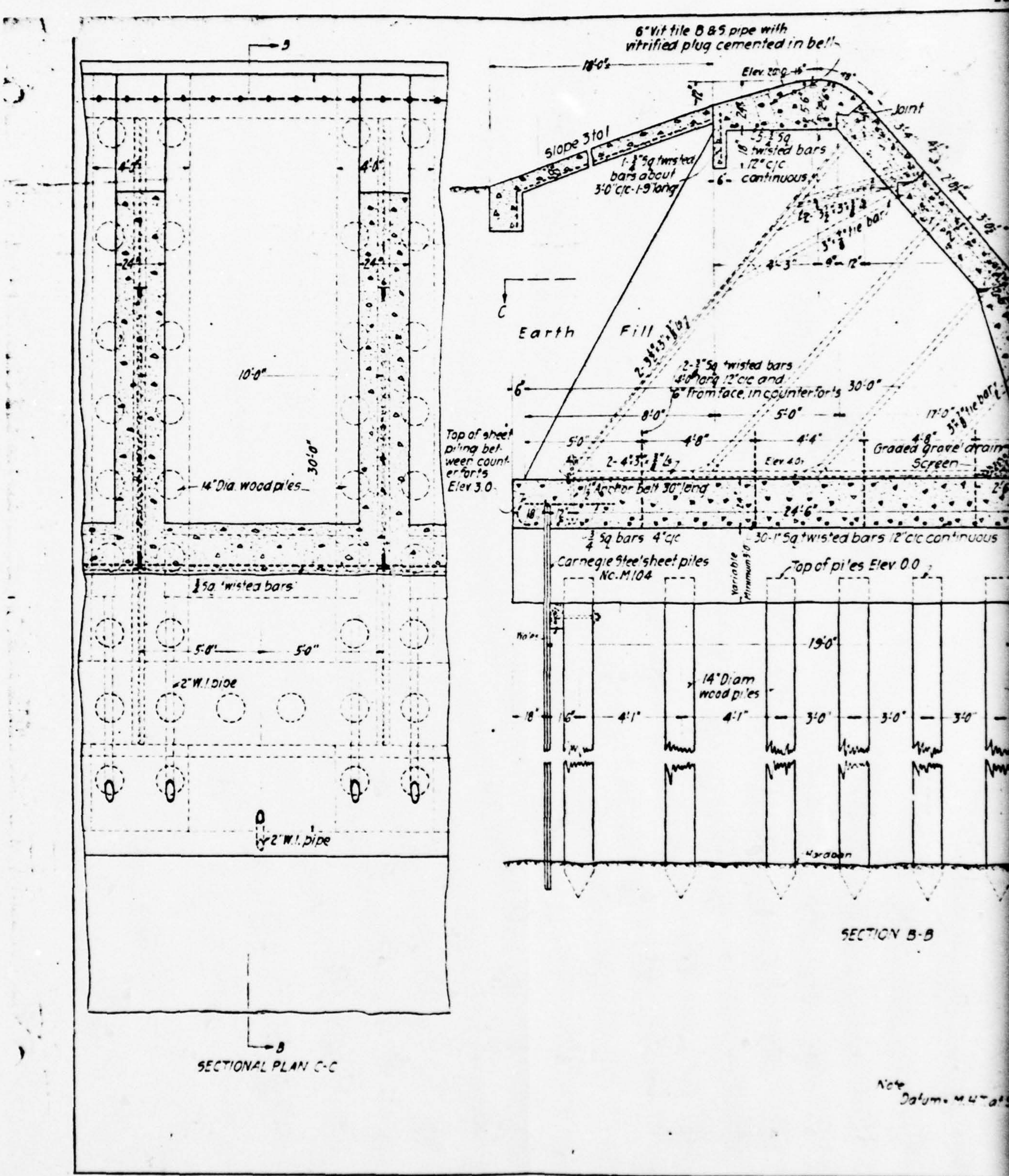
PLATE 1

2

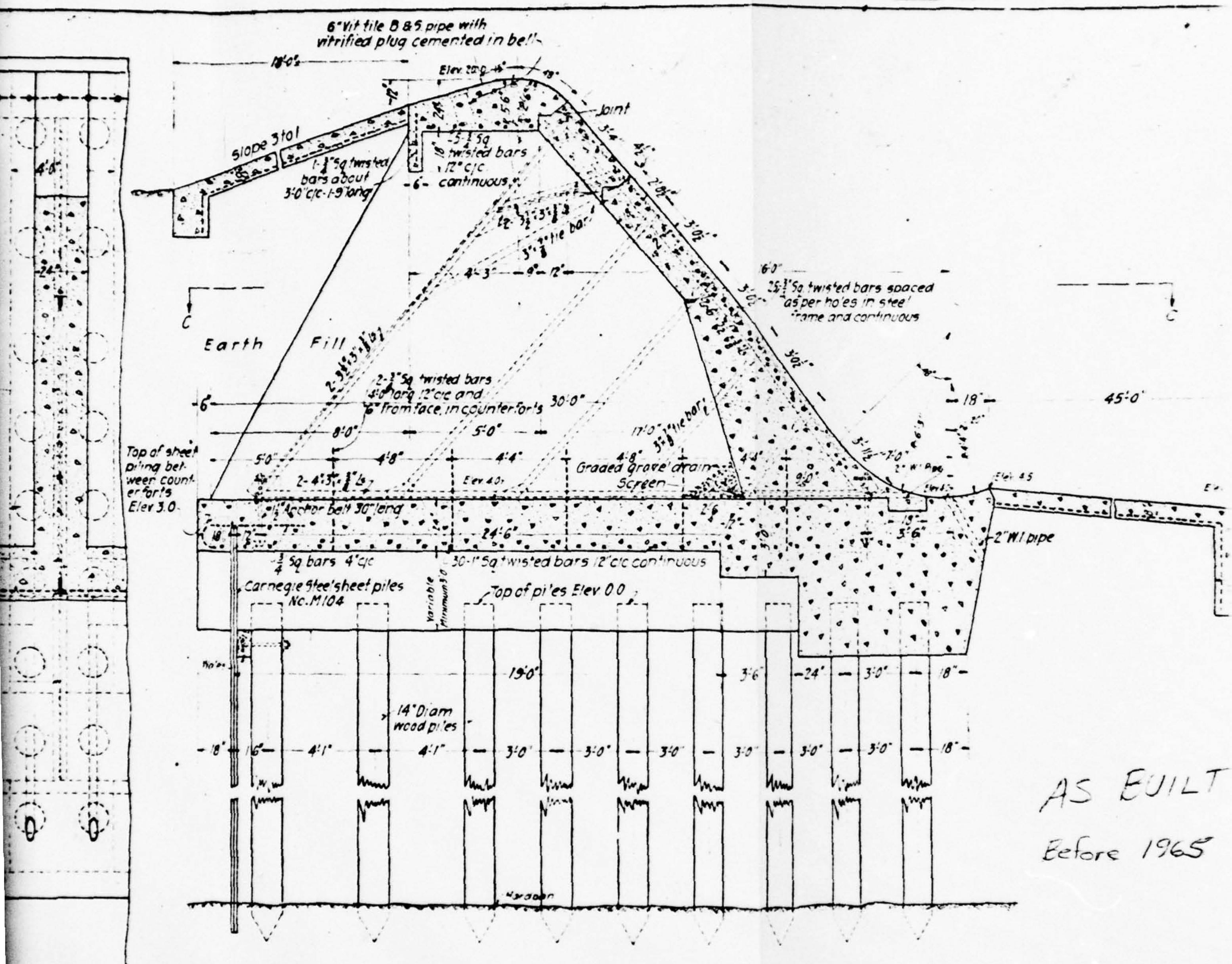
THIS PA
FROM CO




5-3/4 Sq twisted bars continuous 8" c/c
Elev 23.0
Elev 20.0
3-3/4 Sq twisted bars
Gate Stem
W.I. brackets 24" c/c
3-3/4 Sq twisted bars 8'-2" long 12" c/c
3 1/2"x12" Y.P. Stop planks
Elev -0.5
Elev 0.0
Earth Fill
Earth Fill
AS BUILT
Before 1965
SECTION D-D
PLAN E-E
ORADELL DAM
HACKENSACK WATER COMPANY
Note
Datum = M.H.T. at Sandy Hook
PLATE 2


6" Vit tile B & S pipe with vitrified plug cemented in bell
Joint
Slope 3 to 1
Earth Fill
1-1/2" Sq twisted bars about 3'-0" c/c-1'-9" long
5-1/2" Sq twisted bars 12" c/c continuous
2-3/4" Sq twisted bars 4'-0" long 12" c/c and 6" from face in counterforts
Top of sheet piling between counterforts Elev 3.0
Graded gravel drain
Screen
1-1/2" Anchor bolt 30" long
3/4" Sq bars 4" c/c
30-1" Sq twisted bars 12" c/c continuous
Carnegie Steel sheet piles No. M104
Top of piles Elev 0.0
14" Diam wood piles
14" Dia. wood piles
1/2" Sq twisted bars
2" W.I. pipe
SECTION B-B
SECTIONAL PLAN C-C

TICABLE



SECTION B-B

AS BUILT

Before 1965

ORADELL DAM

Note
Datum = M.H.T. at Sandy Hook

PLATE 3

2

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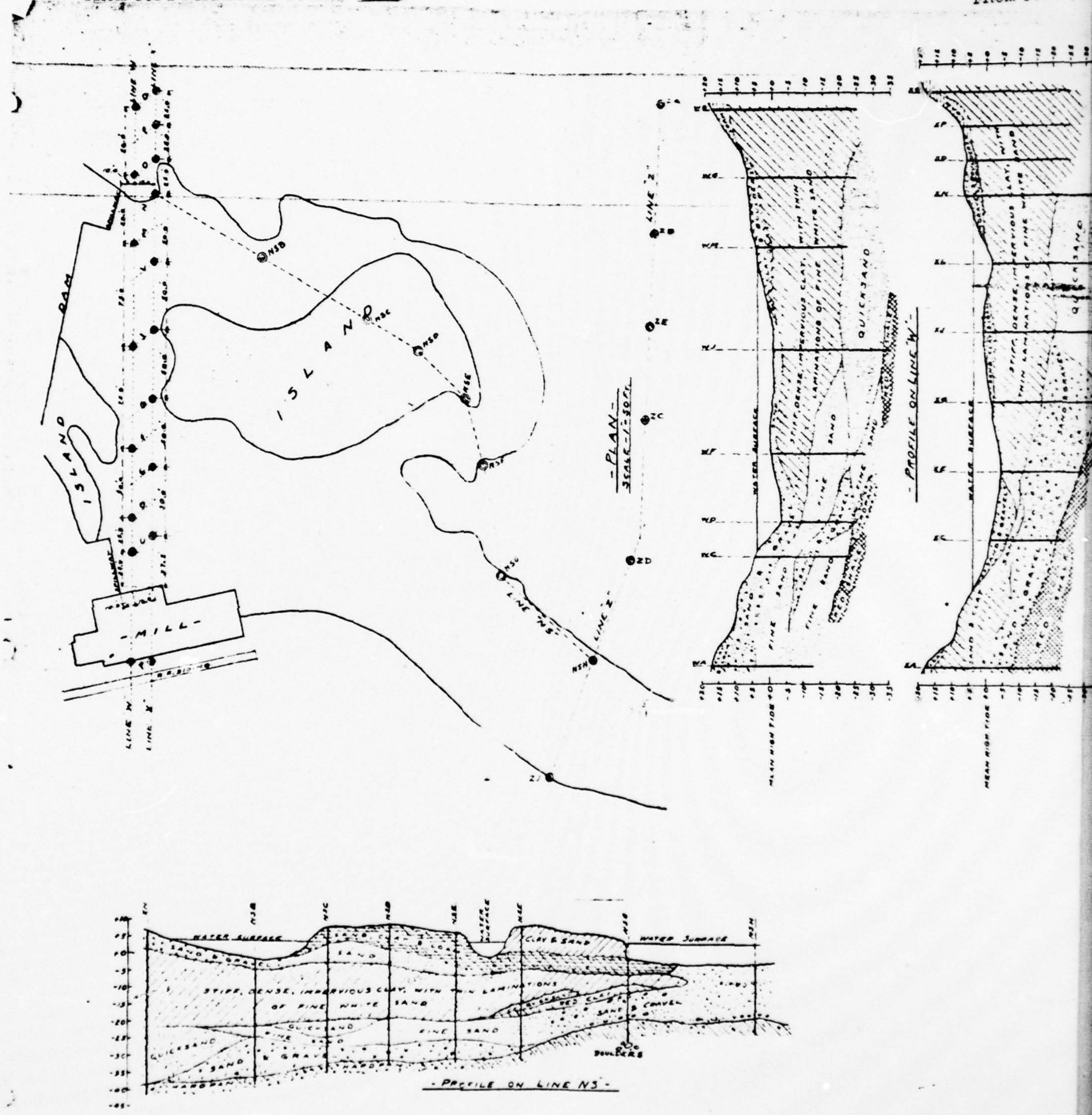

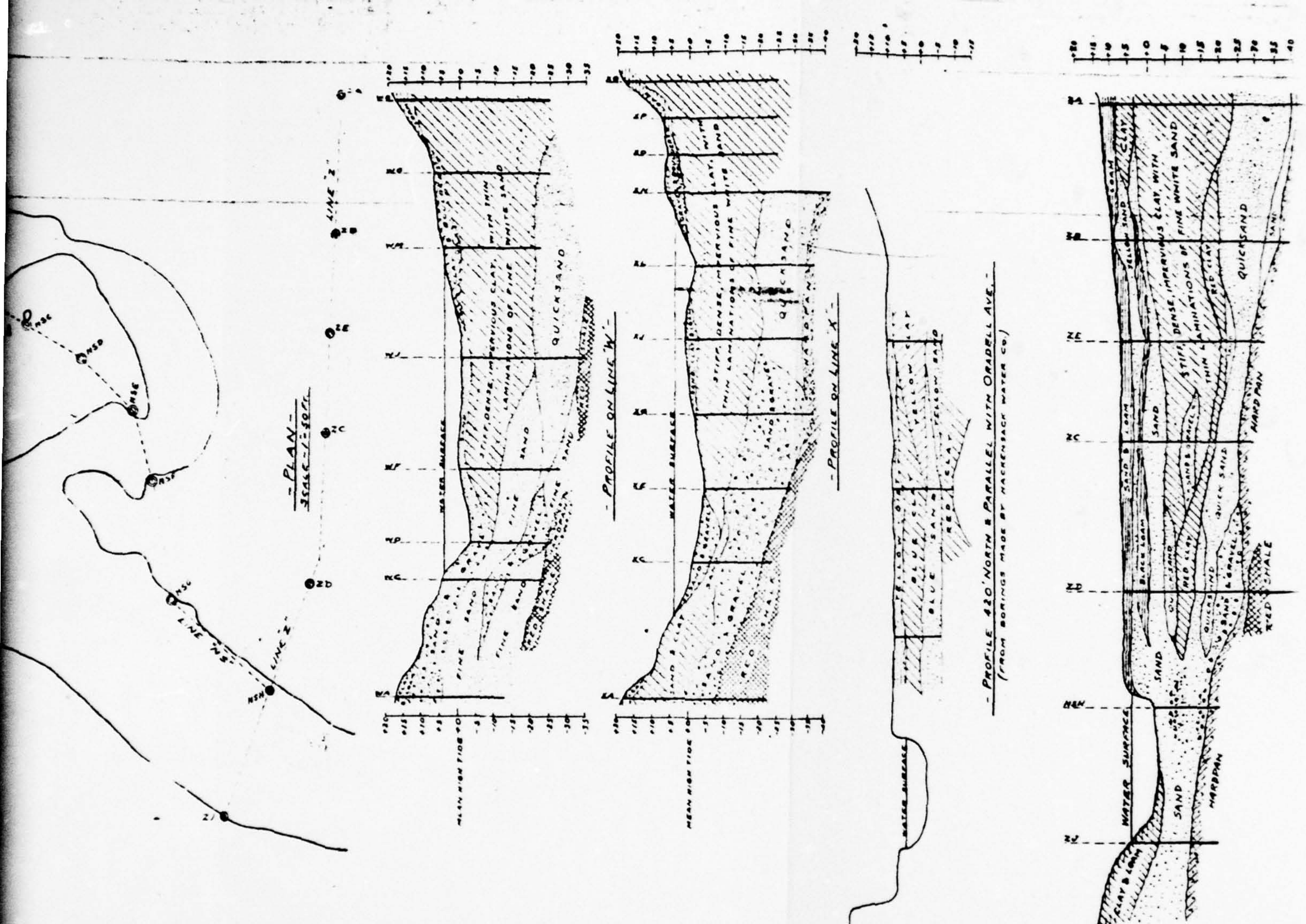

PLATE 4

- DRAWING Nº 48-43.

LOCATION OF TEST BORINGS.
SHOWING STRATIGRAPHY.-
SITE OF PROPOSED ORADELL DAM.
ABOUT 850 FT NORTH OF ORADELL AVE
FOR
HACKENSACK WATER CO.. -

Nicholas S. Hill, Jr.
Consulting Engineer,
112 East 19th Street,
New York, N.Y.

July 14, 1911 -

Checked

2

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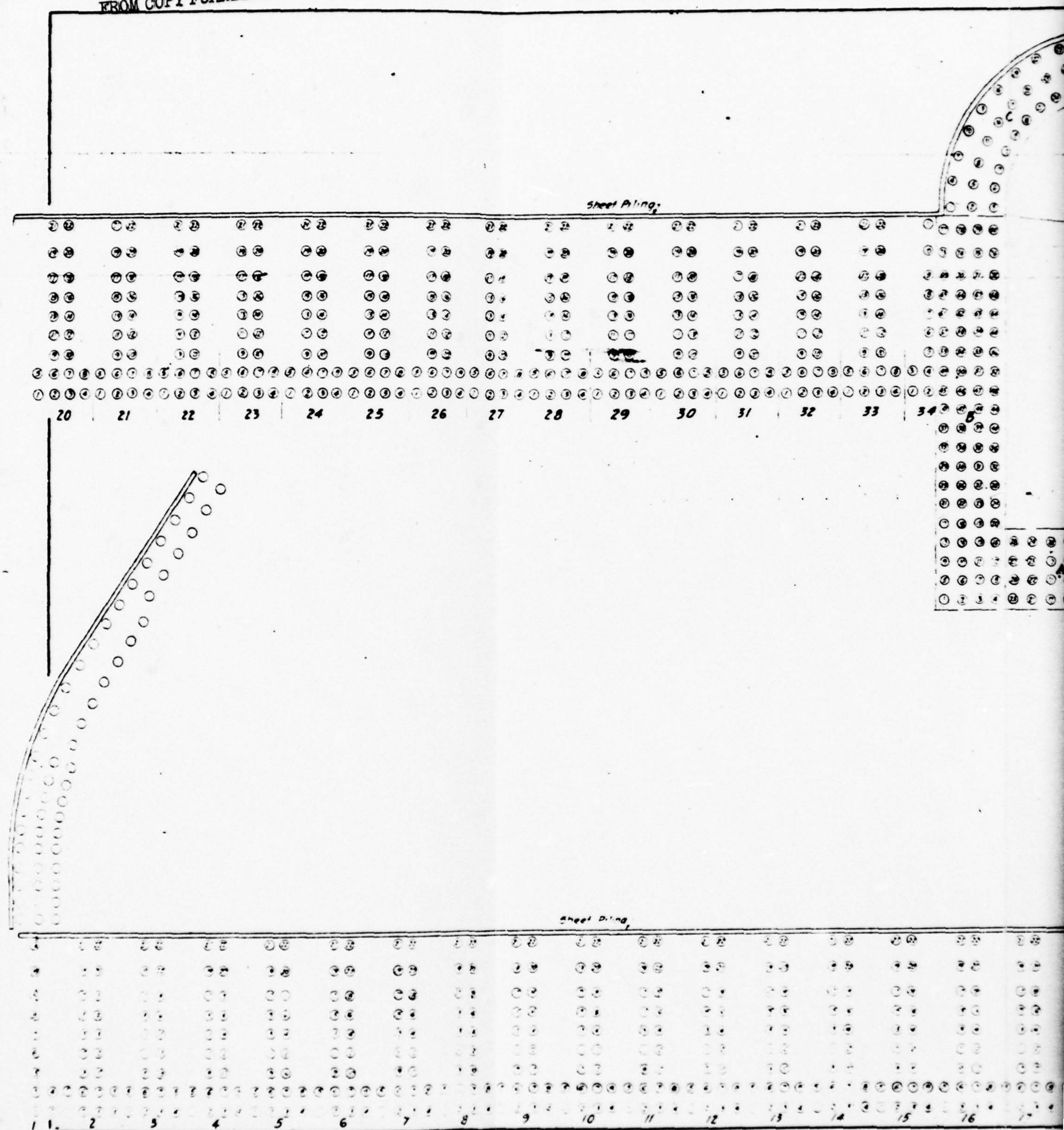

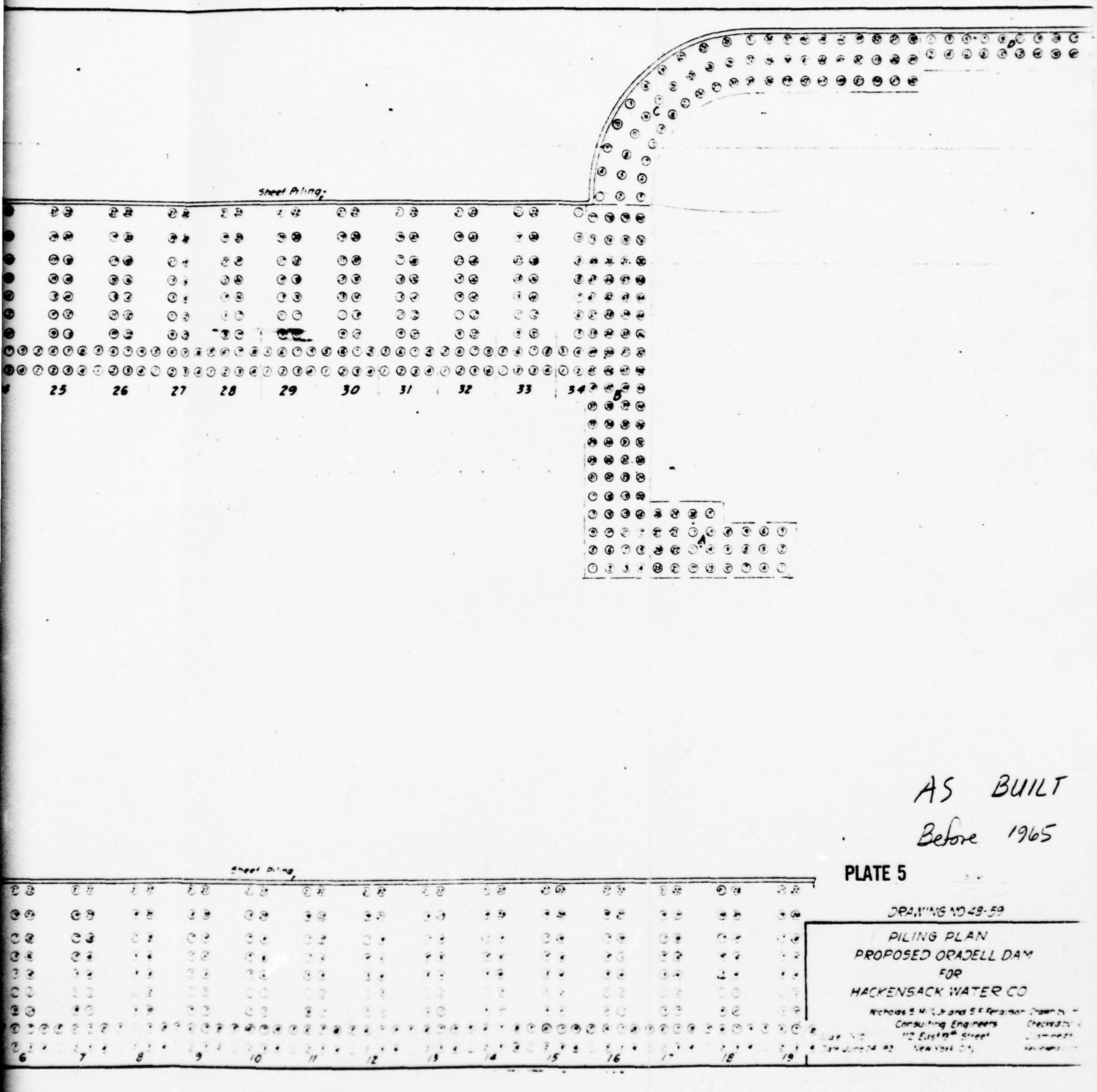

AS BUILT
Before 1965

PLATE 5

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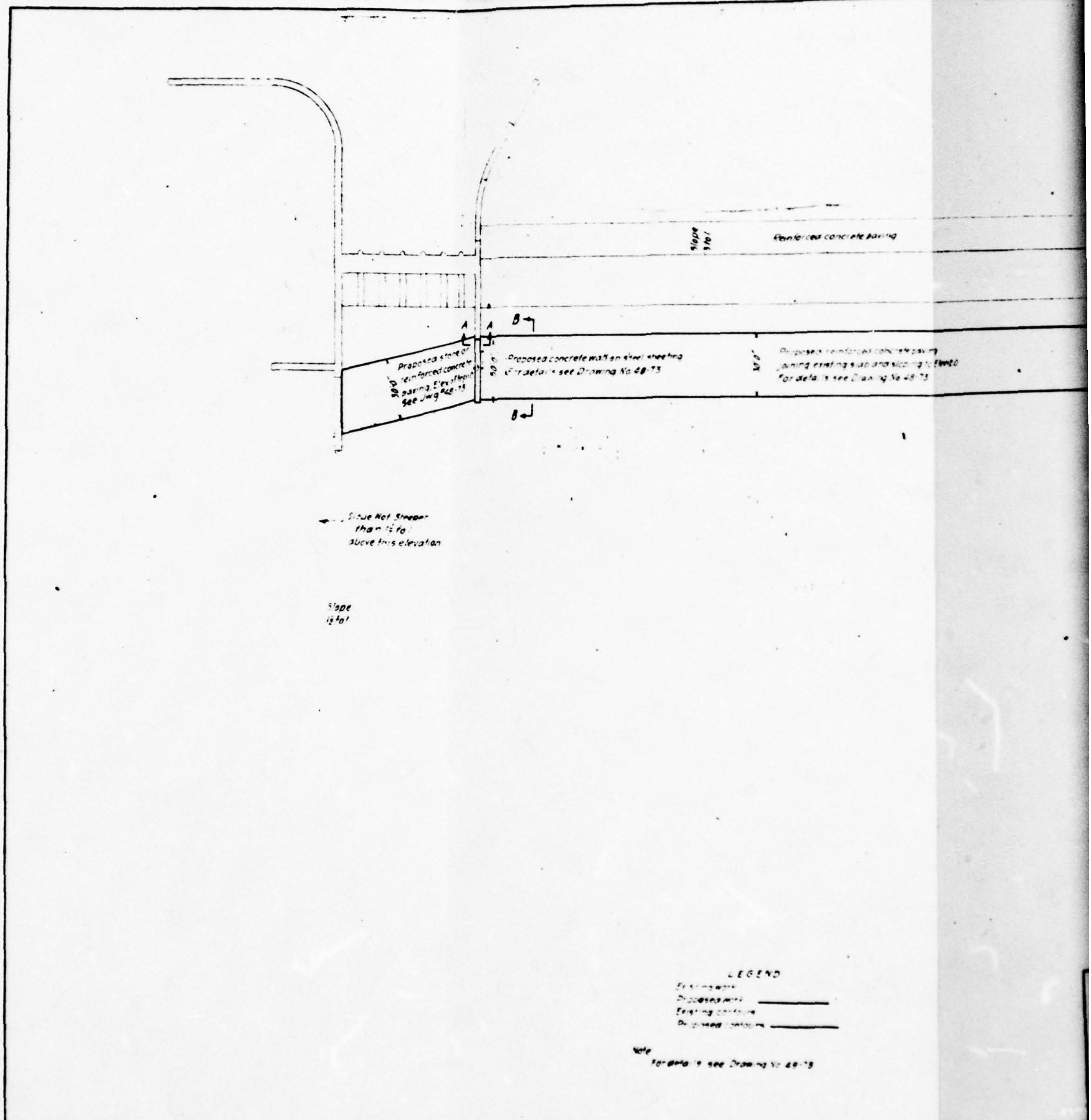

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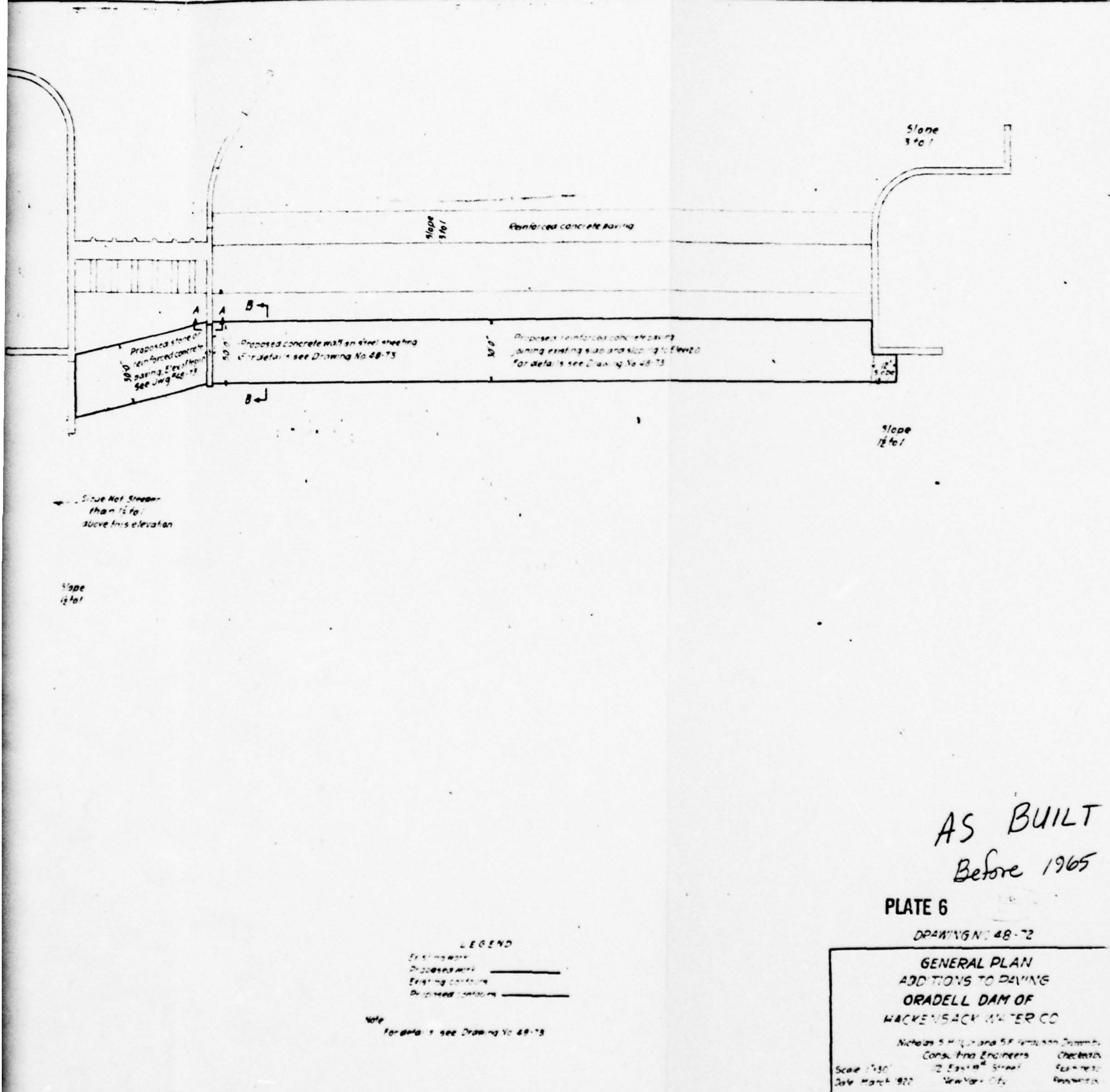
Slope 3 to 1
Slope 3 to 1
Reinforced concrete paving
A
A
B
B
Proposed stone or reinforced concrete paving. Elev. of top see Dwg. 48-73
30.0'
50.0'
Proposed concrete wall on steel sheeting For details see Drawing No. 48-73
30.0'
Proposed reinforced concrete paving joining existing slab and sloping to Elev. 0 For details see Drawing No. 48-73
12" Slope
Slope 1½ to 1
Slope Not Steeper than 1½ to 1 above this elevation
Slope 1½ to 1
LEGEND
Existing work
Proposed work
Existing contours
Proposed contours
Note
For details see Drawing No. 48-73
AS BUILT
Before 1965
PLATE 6
DRAWING NO. 48-72
GENERAL PLAN
ADDITIONS TO PAVING
ORADELL DAM OF
HACKENSACK WATER CO
Consulting Engineers
New York City
Scale
Date March 1922

2

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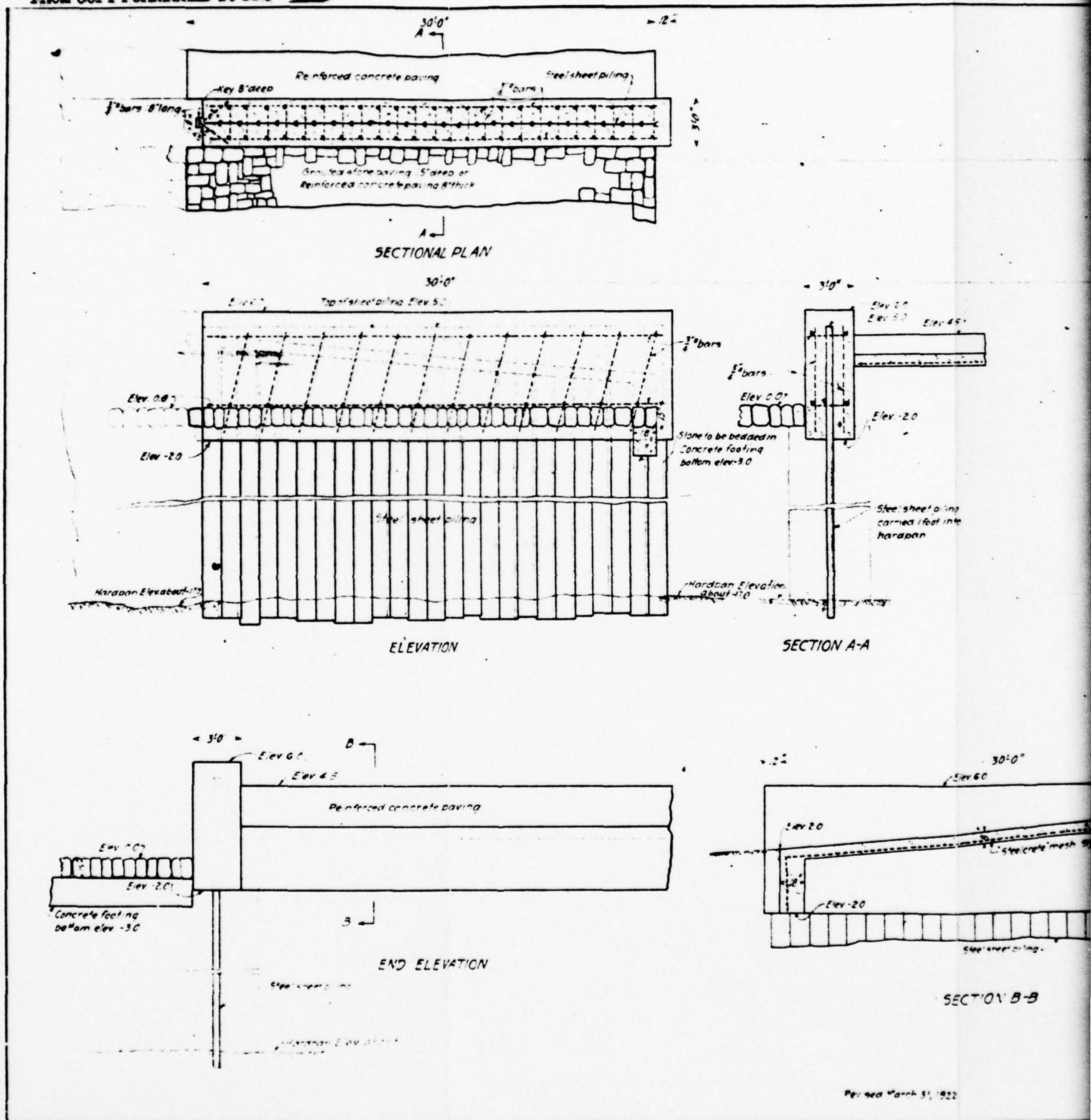
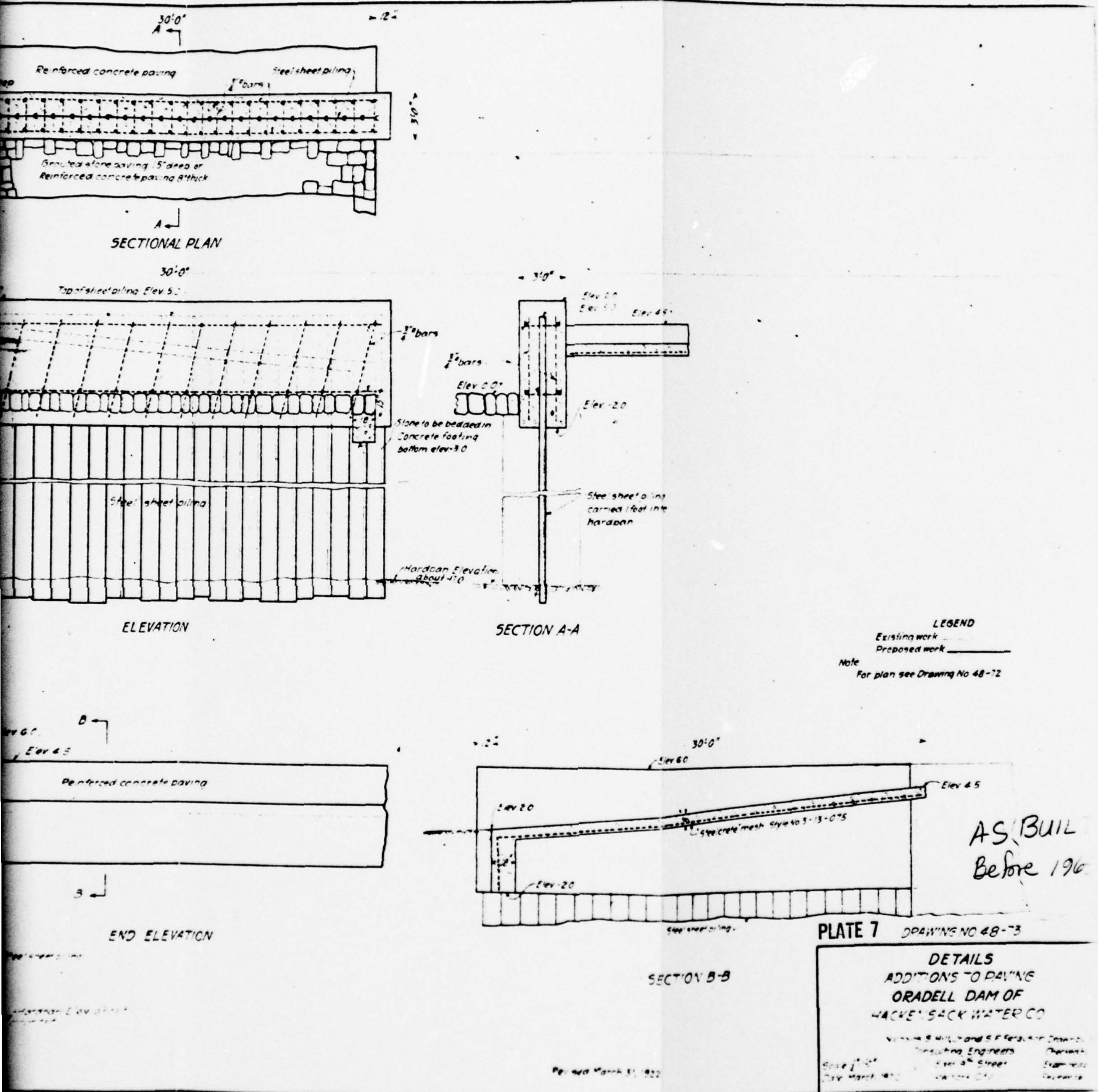
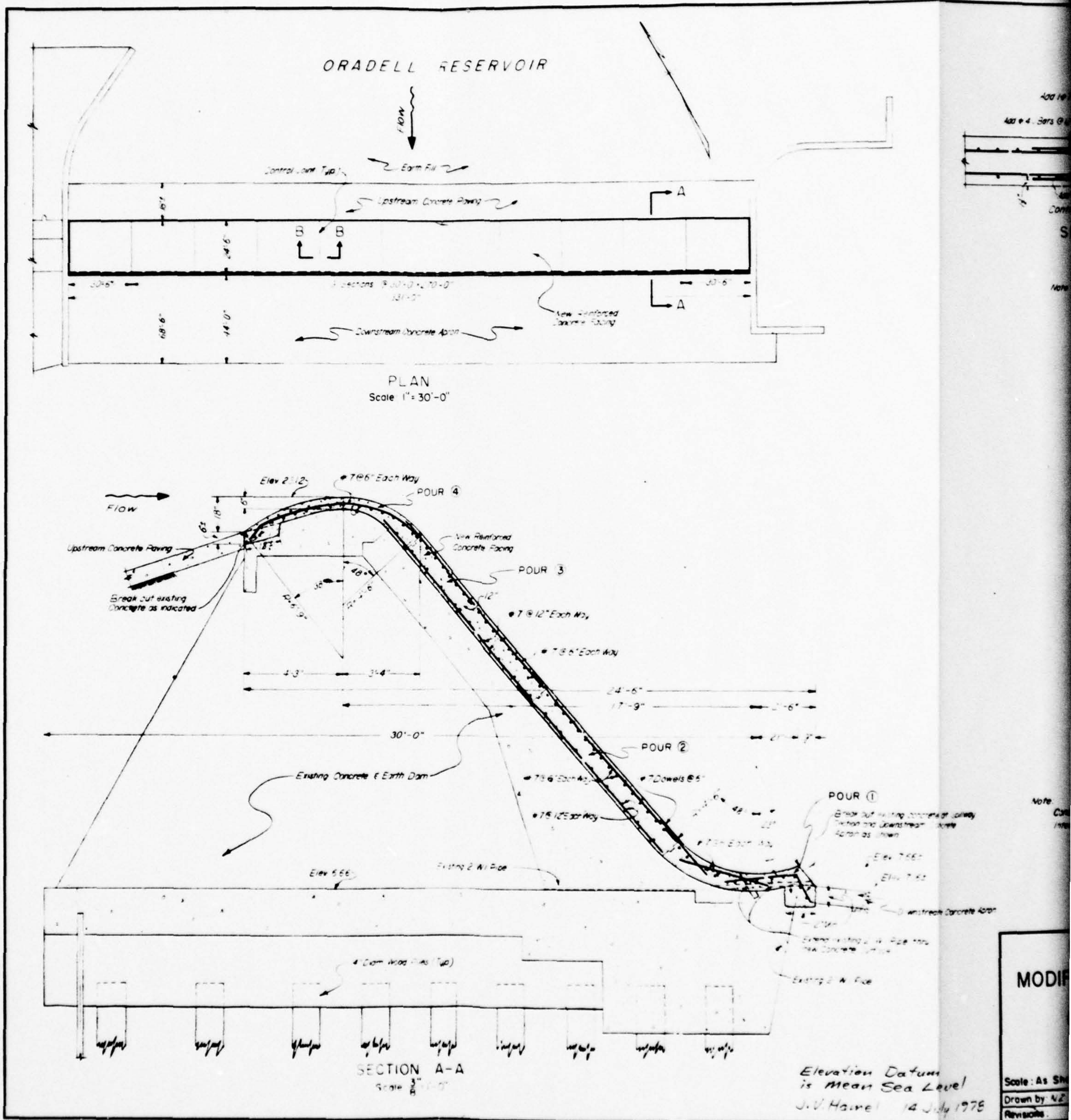



Reinforced concrete paving
Steel sheet piling
Grouted stone paving 15" deep or
Reinforced concrete paving 8" thick
SECTIONAL PLAN
30'0"
Top of sheet piling Elev 5.0
Stone to be bedded in Concrete footing bottom elev -3.0
Steel sheet piling
Hardpan Elevation About -7.0
ELEVATION
Elev 4.5
Elev 0.0
Elev -2.0
Steel sheet piling carried 1 foot into hardpan
SECTION A-A
LEGEND
Existing work
Proposed work
Note
For plan see Drawing No 48-72
Reinforced concrete paving
Elev 4.5
END ELEVATION
Elev 6.0
Elev 4.5
Elev 2.0
Elev -2.0
Steelcrete mesh
Steel sheet piling
SECTION B-B
AS BUILT
Before 196
PLATE 7
DRAWING NO 48-73
DETAILS
ADDITIONS TO PAVING
ORADELL DAM OF
HACKENSACK WATER CO
Consulting Engineers

2



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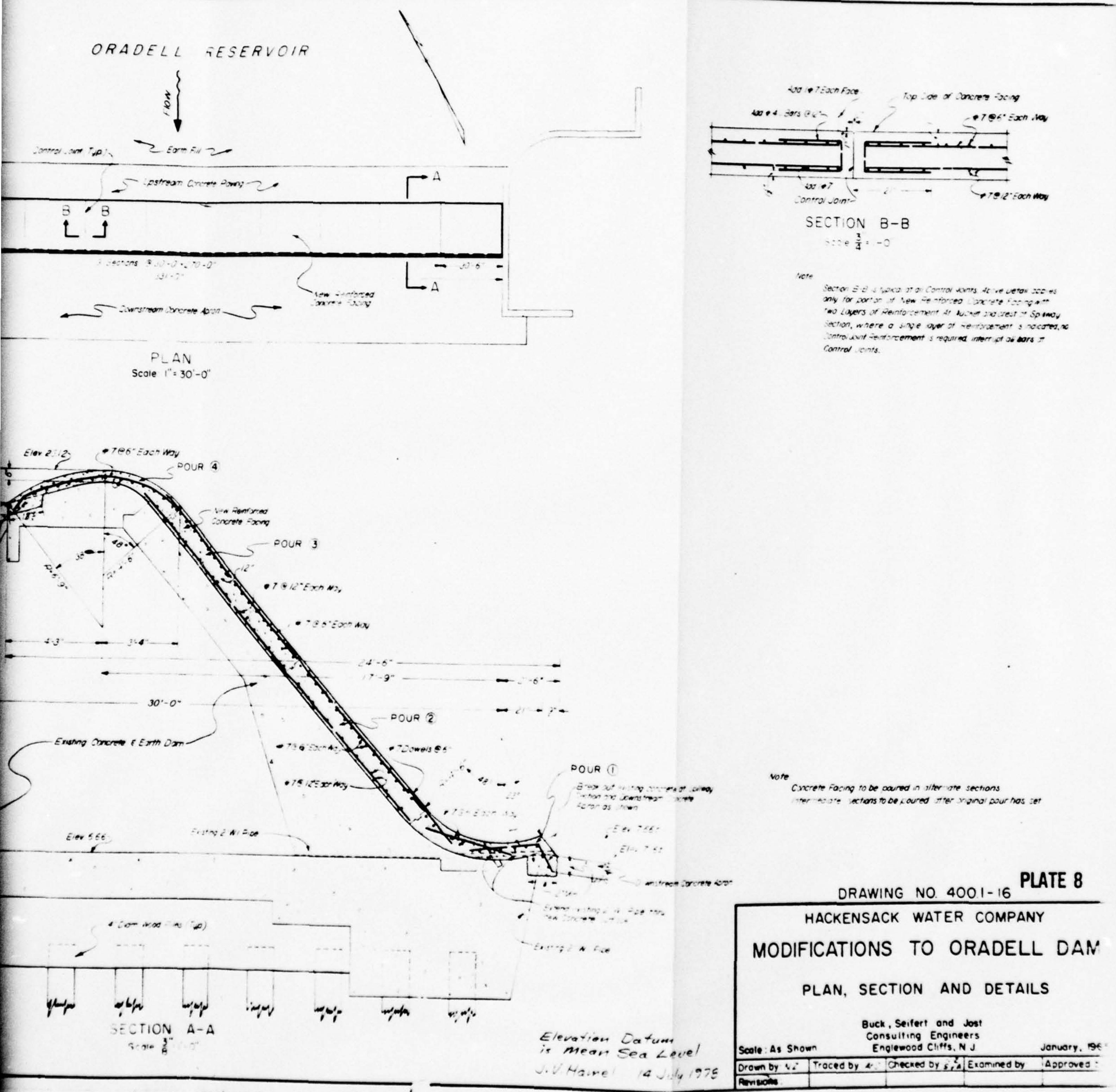

ORADELL RESERVOIR
FLOW
Earth Fill
Control Joint Typ.
Upstream Concrete Paving
B B
A
A
New Reinforced Concrete Facing
Downstream Concrete Apron
PLAN
Scale 1" = 30'-0"
Top Side of Concrete Facing
SECTION B-B
Note
Section B-B is typical at all Control Joints. Above detail applies only for portion of New Reinforced Concrete Facing with Two Layers of Reinforcement. At Bucket and crest of Spillway Section, where a single layer of Reinforcement is indicated, no Control Joint Reinforcement is required. Interrupt all bars at Control Joints.
Elev 23.12
#7 @ 6" Each Way
POUR 4
New Reinforced Concrete Facing
POUR 3
#7 @ 12" Each Way
#7 @ 6" Each Way
24'-6"
17'-9"
30'-0"
POUR 2
Existing Concrete & Earth Dam
#7 Dowels @ 6"
POUR 1
Elev 5.66
Existing 2 W. Pipe
Note
Concrete Facing to be poured in alternate sections. Intermediate sections to be poured after original pour has set.
Downstream Concrete Apron
4" Diam Wood Piles (Typ)
Existing 2 W. Pipe
SECTION A-A
Elevation Datum is Mean Sea Level
J. V. Hawel 14 July 1975
PLATE 8
DRAWING NO. 4001-16
HACKENSACK WATER COMPANY
MODIFICATIONS TO ORADELL DAM
PLAN, SECTION AND DETAILS
Buck, Seifert and Jost
Consulting Engineers
Englewood Cliffs, N.J.
Scale: As Shown
Drawn by
Traced by
Checked by
Examined by
Approved
Revisions

PHOTOGRAPHS

DETAILED PHOTOGRAPH DESCRIPTIONS

Overview Photo - View Upstream (North) at Dam From Right (West) Bank - 12 June 1978

Photo 1 - View Upstream at Dam From Left (East) Bank "Point" Approximately 400 Feet Downstream From Left (East) End of Dam; Sluice Gates in Upper Left Center of Photo; Bank Erosion Area in Lower Left Corner of Photo with Three Feet by Three Feet Rule for Scale - 12 June 1978

Photo 2 - View Downstream (Southeast) Over Crest of Spillway Weir From Approximately 100 Feet Upstream of Right (West) Abutment; Left (East) Abutment at Left Edge of Photo; Sluice Gates at Right Edge of Photo - 12 June 1978

Photo 3 - View East Across Downstream Face of Dam From Right (West) Abutment; Sluice Gates at Left Side of Photo; Spillway in Left Center; Left (East) Abutment Wing Wall in Top Left Center (Close-up in Photo 4) - 12 June 1978

Photo 4 - Close-up of Left (East) Abutment Wing Wall (Photo 3) Showing Cracked and Spalled Concrete and Open Vertical Joint; Three Feet by Three Feet Rule for Scale - 12 June 1978

Photo 5 - View West Across Downstream Face of Dam From Left (East) Abutment; Sluice Gates and Right (West) Abutment Wing Wall (Close-up in Photo 6) in Top Left Corner of Photo - 12 June 1978

Photo 6 - Close-up of Right (West) Abutment Wing Wall (Photo 5) From Training Wall Between Sluice Gate and Spillway Sections (Photo 3) - 12 June 1978

Photo 7 - View Downstream (Southeast) at Sluice Gate Section From Right (West) Abutment; Spillway at Left Edge of Photo - 12 June 1978

Photo 8 - View Upstream (Northwest) at Sluice Gate Section From Downstream End Training Wall Between Sluice Gate and Spillway Sections (Photo 3); Spillway at Right Edge of Photo - 12 June 1978

PHOTO 1

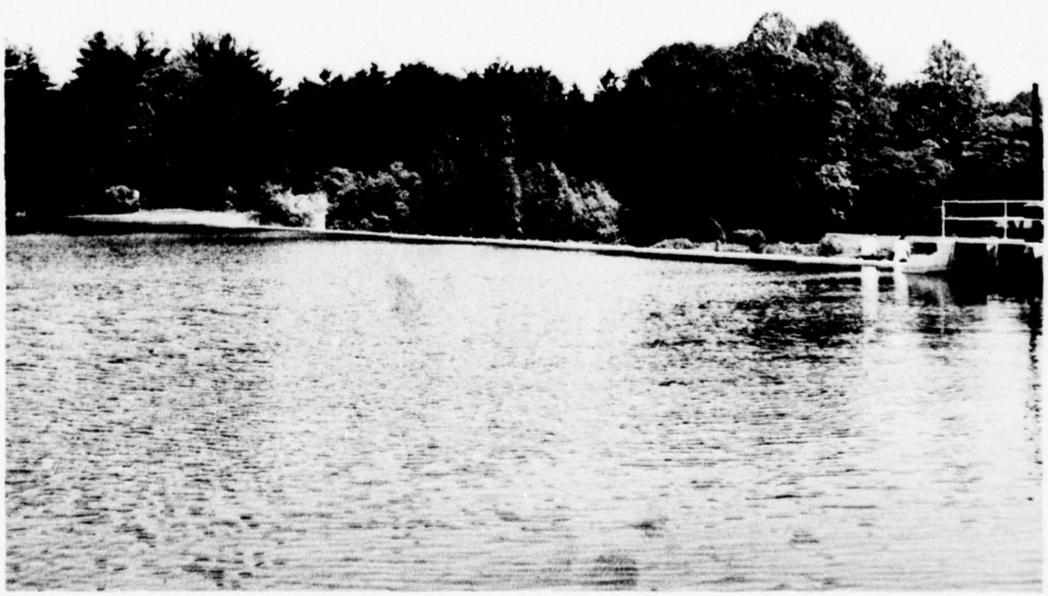


PHOTO 2

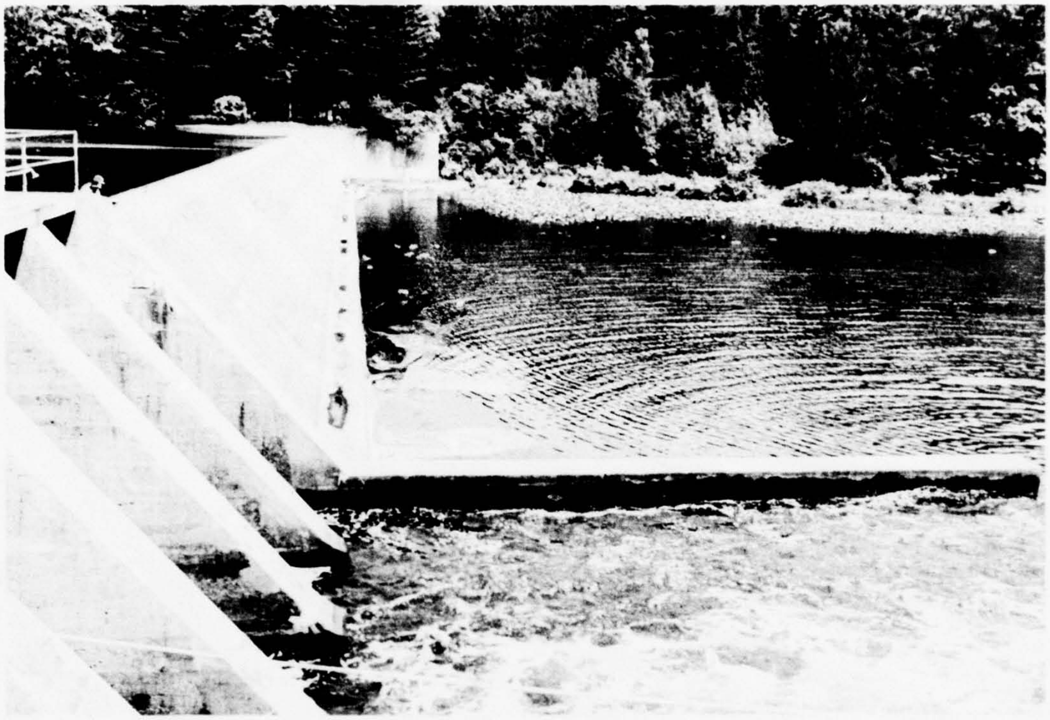


PHOTO 3

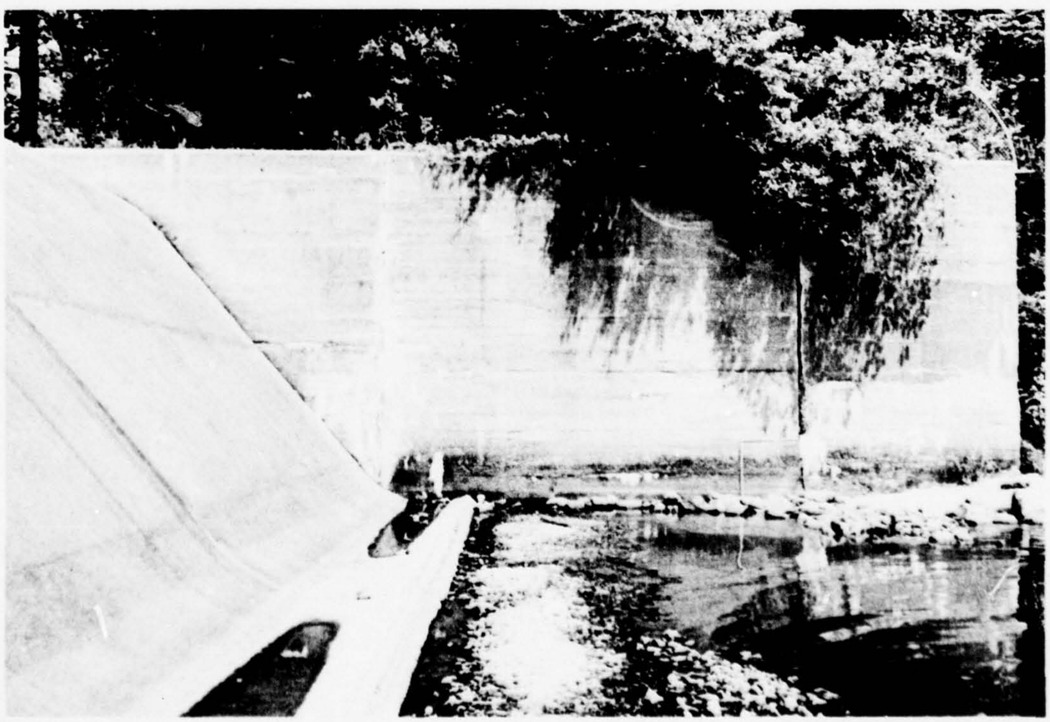


PHOTO 4

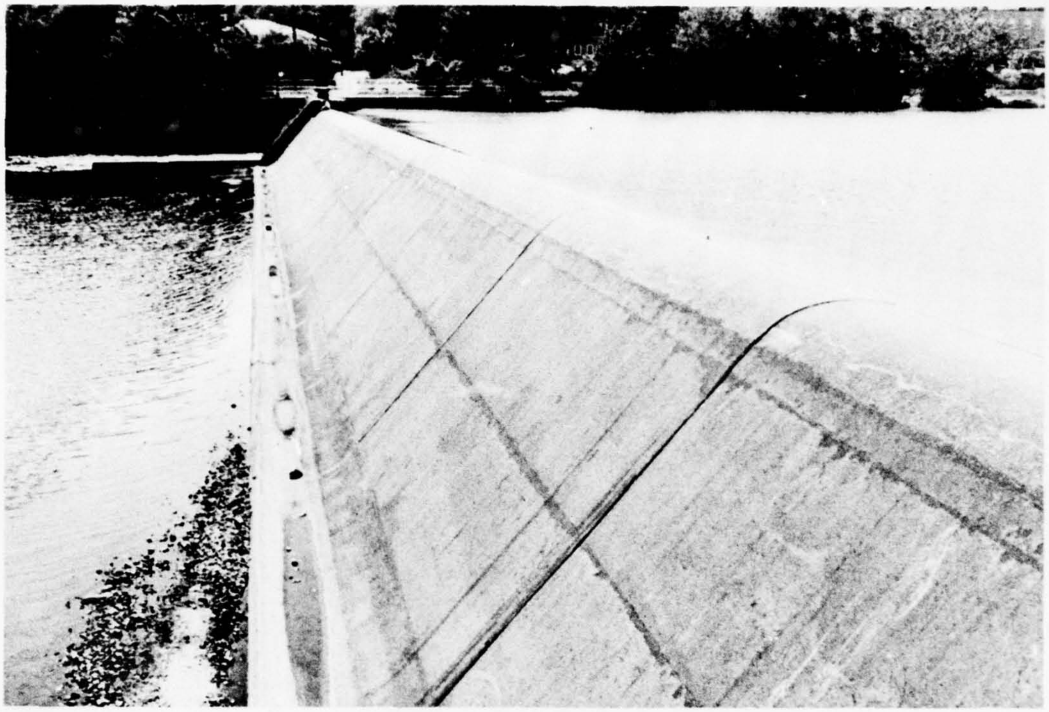


PHOTO 5

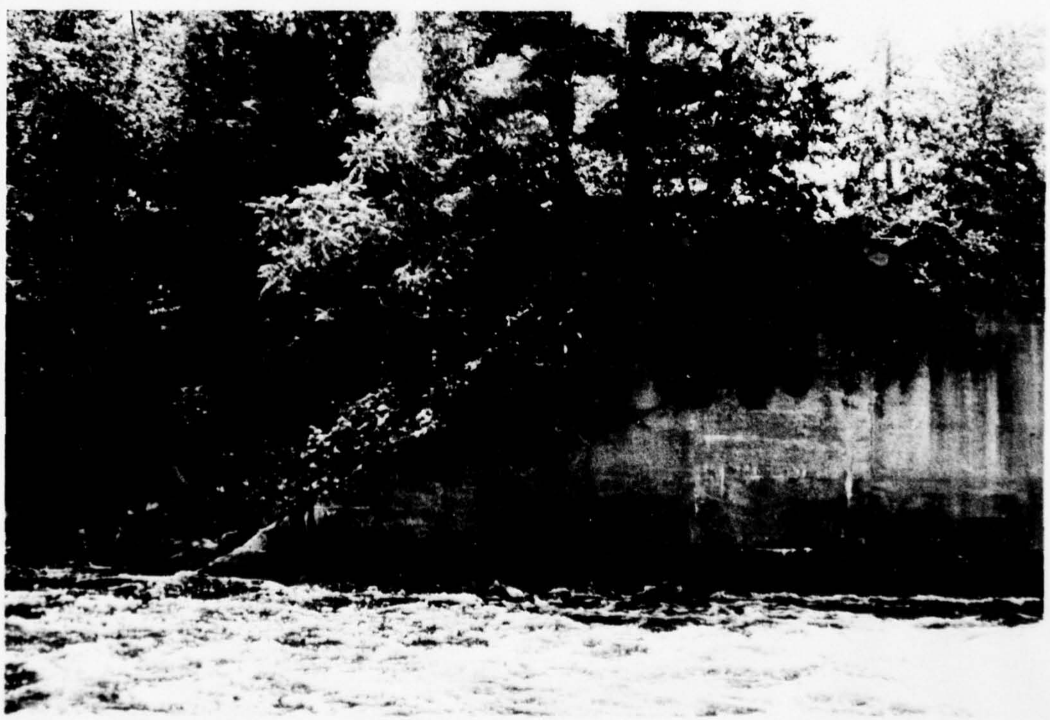


PHOTO 6

PHOTO 7

PHOTO 8

APPENDIX A

CHECK LIST - VISUAL INSPECTION

Check List
Visual Inspection
Phase 1

Name Dam Oradell Reservoir County Bergen State New Jersey Coordinates Lat. 40°57.3' Long. 74°11.7'

Date Inspection 12 June 1978 Weather Sunny, Hot Temperature 85°F.

Pool Elevation at Time of Inspection 23.1 M.S.L. Tailwater at Time of Inspection 7.3 M.S.L.

Inspection Personnel:

MICHAEL BAKER, JR., INC.:

T. J. Dougan
J. V. Hamel
E. U. Gingrich

HACKENSACK WATER CO.
Part-Time During Inspection:

J. J. Cannizzo
J. E. Butler
H. Kahn
B. Willis

E. U. Gingrich Recorder

CONCRETE/MASONRY DAMS

Oradell Reservoir

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SEE PAGE ON LEAKAGE		
STRUCTURE TO ABUTMENT/EMBANKMENT JUNCTIONS	Neoprene sealant deteriorated.	Replace and reseal.
DRAINS	Two (2) two inch diameter horizontal drainpipes were reportedly constructed in base of each panel between counterforts on downstream side of spillway section (total of 66 drainpipes). These drainpipes, which reportedly discharge at approximately El. 7.1, were not observed during the field inspection. The drainpipes were apparently several inches below tailwater and probably covered with sediment. In a subsequent telephone conversation, it was determined that these drainpipes are inspected one or two times per year and cleaned as necessary.	All drainpipes should be inspected periodically and cleaned as necessary to ensure free discharge.
WATER PASSAGES		
FOUNDATION	Not visible.	

CONCRETE/MASONRY DAMS

Oradell Reservoir

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS CONCRETE SURFACES	Longitudinal hairline crack extends from gate section to left abutment along crest of downstream concrete facing at contact with curved weir crest. Crack was inferred to result from tensile stresses at contact.	Hairline crack probably due to flexure at change in curvature of concrete facing. No remedial work required at present time. Recommend periodic visual inspection of concrete.
STRUCTURAL	Dam and spillway are in good condition. A superficial hairline crack runs the entire length of the weir crest, as noted above. A spalled area (one square foot) is located on the weir crest, center of the weir section. Hairline cracks with traces of calcite at several locations on downstream face of weir. Crack one-fourth of an inch in the right divider wall between weir and gate 7--possible undermining or settlement.	Patch spalled area on weir crest.
VERTICAL AND HORIZONTAL ALIGNMENT	Right upstream retaining wall irregular, probably because of poor form work during construction.	
MONOLITH JOINTS	Good condition. Small amount of calcite and iron oxide in second "V" notch, east of right wall--some seepage probably causing this condition.	No immediate work required. Continue to observe for possible maintenance.
CONSTRUCTION JOINTS	Cracking and spalling adjacent to all construction joints on wing walls; calcite is present. Seepage through third construction joint on right wing wall downstream.	Seal and regrout. Provide weep hole to relieve water pressure.

EMBANKMENT

Oradell Reservoir

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS	Note: The embankment (earthfill) section is covered with a concrete facing on the crest and downstream side and partially covered with a concrete facing on the upstream side. The upstream slope of the dam (concrete facing and earthfill portions) was under water at the time of inspection. The embankment (earthfill) section of the dam was therefore neither visible nor accessible at the time of inspection.	
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE		
SLOUGHING OR EROSION OF EMBANKMENT AND ABUTMENT SLOPES		
VERTICAL AND HORIZONTAL ALIGNMENT OF THE CREST		
RIPRAP FAILURES		

Oradell Reservoir

EMBANKMENT
(See Note on Sheet 1)

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
JUNCTION OF EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM		
ANY NOTICEABLE SEEPAGE		
STAFF GAGE AND RECORDER	None	
DRAINS	See comments on "DRAINS"--Sheet 1 of "CONCRETE/MASONRY DAMS".	

OUTLET WORKS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	One foot wide spalled area above gates 3, 4, 5 and 7. Calcite present on and above all gate walls.	Continue observations until areas warrant reconstruction or other remedial work.
INTAKE STRUCTURE	Mostly underwater; no problems were visible.	
OUTLET STRUCTURE	Gates number 1 and 2 were inoperable. Spalled areas at toes of gates 3, 4, 5 and 7--not serious. One-fourth of an inch crack, three feet down, adjacent to gate 7. Slots for the insertion of stop planks are located on the upstream side of the sluice gate chambers. These stop planks can be used when inspection or repairs of the sluice gates or sluice gate chambers are necessary. Each sluice gate chamber can be individually unwatered by using the stop planks as a cutoff of the reservoir flow. The stop planks are currently stored in a maintenance shed at the dam.	The gates should be repaired very soon. No structural significance; cosmetic repair as appropriate.
OUTLET CHANNEL	Sediment and vegetation in area extending about 100 feet downstream from weir along left bank. Localized areas of riprap failure and bank erosion along left bank from dam to "point" about 400 feet downstream from dam and around this "point" to Oradell Avenue Bridge. Also localizes areas of riprap failure and bank erosion along right bank from dam to bridge.	Place riprap (with underlying graded filters as necessary) in scour areas along left bank (especially "point" area) and along right bank to Oradell Avenue Bridge.
EMERGENCY GATE	No emergency gate.	

UNGATED SPILLWAY

Oradell Reservoir

(See "CONCRETE/MASONRY DAMS" Sheet 2)

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE WEIR		
APPROACH CHANNEL		
DISCHARGE CHANNEL		
BRIDGE AND PIERS		

GATED SPILLWAY

Oradell Reservoir

(See "OUTLET WORKS" Sheet)

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE SILL		
APPROACH CHANNEL		
DISCHARGE CHANNEL		
BRIDGE AND PIERS		
GATES AND OPERATION EQUIPMENT		

INSTRUMENTATION

Oradell Reservoir

VISUAL EXAMINATION	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
MONUMENTATION/SURVEYS	U.S.G.S. Bench Mark--1.5 inch diameter brass plate, El. 25.80--on concrete retaining wall along right abutment upstream of sluice gate section.	
OBSERVATION WELLS	None	
WEIRS	None	
PIEZOMETERS	None	
OTHER	Water level gage to M.S.L. near the bench mark on the right abutment retaining wall. Gates manually operated to adjust to variations in intake gaged at treatment plant downstream.	

RESERVOIR

Oradell Reservoir

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SLOPES	Relatively flat and well vegetated, generally fine grained glaciofluvial soils with moderate to high erosion potential.	
SEDIMENTATION	Minor sedimentation at upper end of reservoir--as would be expected.	No significant erosion or sedimentation problems observed in the reservoir.

DOWNSTREAM CHANNEL

Oradell Reservoir

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION (OBSTRUCTIONS, DEBRIS, ETC.)	Minor sedimentation and aquatic vegetation along left (east) bank for distance about 100 feet downstream from left end of dam in slack water area upstream of "point". Localized areas of riprap failure and bank erosion on both banks from dam to Oradell Avenue Bridge (See comments on "OUTLET CHANNEL" of the "OUTLET WORKS" Sheet).	Sediment and aquatic vegetation will be scoured away during flood flow.
SLOPES	Downstream channel slopes low and flat--generally fine grained glaciofluvial soils with moderate to high erosion potential.	
APPROXIMATE NO. OF HOMES AND POPULATION	Reach 1 extends approximately 800 feet downstream from dam to Oradell Avenue Bridge (clear opening approximately 90.0 feet wide by 5.5 feet high) through park like area. There are no homes located below dam crest level in Reach 1. Reach 2 extends approximately 2900 feet downstream from Oradell Avenue Bridge to another bridge. There are an estimated 20 homes and 100 people, and the water intake and treatment facilities of the Hackensack Water Co., below dam crest level in Reach 2. Reach 3 extends approximately 4100 feet further downstream to a third bridge connecting River Edge with New Milford. There are an estimated 50 houses and 300 people in low lying portions of Reach 3.	

APPENDIX B

CHECK LIST - ENGINEERING DATA

CHECK LIST
ENGINEERING DATA
DESIGN, CONSTRUCTION, OPERATION

Oradell Reservoir

ITEM	REMARKS
PLAN OF DAM	Reference drawings: "Proposed Oradell Dam for Hackensack Water Co." prepared by N.S. Hill, Jr., Consulting Engineer, New York, New York 1911-1923 (numerous sheets, some of which are included as Plates 1-7 of this report); "Modifications to Oradell Dam--Plan, Section and Details," Drawing No. 400.1-16 prepared by Buck, Seifert and Jost Consulting Engineers, Englewood Cliffs, New Jersey, January 1965 (included as Plate 8 of this report). Plan of Dam--See Plates 1, 4, 5, 6 and 8 of this report.
REGIONAL VICINITY MAP	Section of U.S.G.S. Hackensack, NJ 7.5 min. Quadrangle in this report as the Location Plan.
CONSTRUCTION HISTORY	Refer to: Hill, N.S., Jr., "The Oradell Dam of the Hackensack Water Company," Transactions, American Society of Civil Engineers (A.S.C.E.), Vol. 89, 1926, pp. 1181-1202 (includes plan and section drawings which are included as Plates 1, 2 and 3 in this report).
TYPICAL SECTIONS OF DAM	Hill's 1926 A.S.C.E. paper, plus Plates 2, 3 and 8 of this report and Hill's Drawing No. 48-58 not included in this report.
HYDROLOGIC/HYDRAULIC DATA	Hill's 1926 A.S.C.E. paper and data furnished by Hackensack Water Co. (report by consultant to Hackensack Water Co. to be available in near future). See also microfiche files of N.J.D.E.P.
OUTLETS - PLAN	Hill's 1926 A.S.C.E. paper, Plates 1, 2 and 6 of this report and Hill's Drawings No. 48-44, 48-47 and 48-48 not included in this report.
- DETAILS	Hill's 1926 A.S.C.E. paper, Plate 2 of this report and Hill's Drawing No. 48-53 not included in this report.
- CONSTRAINTS	None observed.
- DISCHARGE RATINGS	Sluice gate discharge rating may be available from Hackensack Water Co. Spillway discharge rating table on attached sheet.
RAINFALL/RESERVOIR RECORDS	Rainfall is recorded by Hackensack Water Co. at New Milford Gaging Station located one-half mile downstream from dam. Rainfall records are available at U.S.G.S. office in Trenton, New Jersey. Rainfall and reservoir records are available from the Hackensack Water Co.

Oradell Reservoir

ITEM	REMARKS
DESIGN REPORTS	Refer to Hill's 1926 A.S.C.E. paper and discussions of that paper on pp. 1203-1212 of A.S.C.E. Transactions.
GEOLOGY REPORTS	Some geologic and geotechnical information included in Hill's 1926 A.S.C.E. paper and on Plate 4 of this report.
DESIGN COMPUTATIONS HYDROLOGY & HYDRAULICS DAM STABILITY SEEPAGE STUDIES	Refer to Hill's 1926 A.S.C.E. paper and discussions following it. Also reports and data in microfiche files of N.J.D.E.P., including Board of Consultants "Report on Oradell Dam" 1 December 1921.
MATERIALS INVESTIGATIONS BORING RECORDS LABORATORY FIELD	See note above on "GEOLOGY REPORTS". No laboratory or field test data are readily available on materials (soils, rock, concrete) used or encountered in dam construction. It is not known whether tests were indeed made on these materials.
POST-CONSTRUCTION SURVEYS OF DAM	None available
BORROW SOURCES	According to Hill's 1926 A.S.C.E. paper (p. 1189), the earthfill used in the dam was "hydraulically placed and of stiff clay". This fill material was probably obtained from dredging operations in the reservoir area.

Oradell Reservoir

ITEM	REMARKS
MONITORING SYSTEMS	Stream and rain gauges located at New Milford, New Jersey 0.6 miles downstream of dam. Data obtained by Hackensack Water Co. for the U.S.G.S. U.S.G.S. Bench Mark El. 25.8 at damsite.
MODIFICATIONS	Sluice gate operation was changed from manual to electrical in 1959. Some repairs were made to downstream face of dam in 1960. An eight inch reinforced concrete mat was placed on the downstream face of the dam spillway in 1965. (See Plate 8 of this report).
HIGH POOL RECORDS	Highest known pool elevation reached 24.8 feet, (M.S.L. datum) 9 November 1977. Additional pool records are available from Hackensack Water Co.
POST-CONSTRUCTION ENGINEERING STUDIES AND REPORTS	Engineering inspection of Oradell Dam performed on 22 April 1968 by G. M. Haskew, Jr., Vice President and Chief Engineer, Hackensack Water Co. Copies of this report are available in microfiche files of N.J.D.E.P. and from Hackensack Water Co. "Application for Permit for Construction on Repair of Dam" 25 November 1964 (spillway resurfacing)--available in microfiche files of N.J.D.E.P. and from Hackensack Water Co.
PRIOR ACCIDENTS OR FAILURE OF DAM DESCRIPTION REPORTS	None
MAINTENANCE OPERATION RECORDS	Available from Hackensack Water Co.

Oradell Reservoir

ITEM		REMARKS
SPILLWAY PLAN		Plates 1, 6 and 8 of this report.
	SECTIONS	Plates 1, 3 and 8 of this report and Hill's Drawing No. 48-58, not included in this report.
	DETAILS	Plates 3, 7 and 8 of this report.
OPERATING EQUIPMENT PLANS & DETAILS		Hill's Drawing No. 48-53, not included in this report. Contact Hackensack Water Co. for additional information.

Note: Considerable pertinent engineering information was included in or with letters of 9 and 23 June 1978 from J. J. Cannizzo, Director, Engineering Design and Construction, Hackensack Water Company, to C. Y. Chen, Chief Geotechnical Engineering, Michael Baker, Jr., Inc.

ORADELL DAM

RATES OF DISCHARGE OVER SPILLWAY SECTION

Elevations are referenced to M.S.L. Datum; flows are in M.G.D.

Elevation	Flow	Elevation	Flow	Elevation	Flow	Elevation	Flow
23.16	0	23.31	41.3	23.46	116.9	23.61	214.8
23.17	0.7	23.32	45.5	23.47	122.8	23.62	222.0
23.18	2.0	23.33	49.9	23.48	128.8	23.63	229.3
23.19	3.7	23.34	54.4	23.49	134.9	23.64	236.6
23.20	5.7	23.35	58.9	23.50	141.1	23.65	244.7
23.21	8.0	23.36	63.6	23.51	147.4	23.66	251.6
23.22	10.5	23.37	68.4	23.52	153.7	23.67	259.2
23.23	13.2	23.38	73.4	23.53	160.2	23.68	266.8
23.24	16.1	23.39	78.5	23.54	166.7	23.69	274.9
23.25	19.1	23.40	83.7	23.55	173.3	23.70	282.4
23.26	22.5	23.41	85.9	23.56	180.0	23.71	290.3
23.27	26.0	23.42	94.4	23.57	186.8	23.72	298.2
23.28	29.6	23.43	99.8	23.58	193.7	23.73	306.8
23.29	33.4	23.44	105.5	23.59	200.7	23.74	314.3
23.30	37.3	23.45	111.2	23.60	207.7	23.75	321.9

Note: This sheet was included by J. J. Cannizzo of Hackensack Water Company in his letter of 9 June 1978 to C. Y. Chen of Michael Baker, Jr., Inc.

CHECK LIST
HYDROLOGIC AND HYDRAULIC DATA
ENGINEERING DATA

DRAINAGE AREA CHARACTERISTICS: 112 square miles of relatively flat terrain

ELEVATION TOP NORMAL POOL (STORAGE CAPACITY): 23.16 (10,026 acre-feet)

ELEVATION TOP FLOOD CONTROL POOL (STORAGE CAPACITY): Not Applicable

ELEVATION MAXIMUM DESIGN POOL: 23.6 (original design)

ELEVATION TOP DAM: 25.6 - top of sluice gate operating platform

CREST:

a. Elevation 23.16 - crest of spillway weir since 1965; ("as built" crest elevation)
b. Type Earthfill embankment with concrete envelope--ogee weir.
c. Width Rounded crest on spillway weir (see drawings).
d. Length 402 feet (includes 331 feet spillway weir).
e. Location Spillover Across river channel
f. Number and Type of Gates Ungated spillway; sluice gates discussed below.

OUTLET WORKS:

a. Type seven sluice gates, each 7.0 by 9.0 feet.
b. Location Right (west) end of dam.
c. Entrance inverts El. 2.1 feet (M.S.L. datum)
d. Exit inverts El. 2.6 feet (M.S.L. datum)
e. Emergency draindown facilities Sluice gates

HYDROMETEOROLOGICAL GAGES:

a. Type Not readily available.
b. Location 0.6 miles downstream of dam at water co. intake (New Milford Gaging Station No. 01378500).
c. Records Obtained by Hackensack Water Co and reported to U.S.G.S.--Trenton, New Jersey

MAXIMUM NON-DAMAGING DISCHARGE Not Available

APPENDIX C

"INVESTIGATION OF SAFETY OF WOODCLIFF LAKE, ORADELL, AND LAKE TAPPAN DAMS OF HACKENSACK WATER COMPANY AND LAKE DE FOREST DAM OF SPRING VALLEY WATER COMPANY"

DATED JULY, 1978

BY

Buck, Seifert, and Jost, Inc.
Consulting Engineers
Englewood Cliffs, New Jersey

Prepared For

Hackensack Water Company
4100 Park Avenue
Weehawken, New Jersey 07087

Note: Portions of the above report pertaining to Oradell Dam were photocopied by the Hackensack Water Comany and forwarded to Michael Baker, Jr., Inc., in August 1978 in cooperation with the Phase I Inspection of Oradell Dam.

INVESTIGATION

of

SAFETY

of

WOODCLIFF LAKE, ORADELL, AND LAKE TAPPAN DAMS

of

HACKENSACK WATER COMPANY

and

LAKE DE FOREST DAM

of

SPRING VALLEY WATER COMPANY

BUCK, SEIFERT AND JOST, INC.
Consulting Engineers
Englewood Cliffs, New Jersey

JULY 1978

Lake DeForest Dam

This dam is conservatively designed and is in excellent shape.

a. A "V" notch weir should be installed in place of the rectangular weir at the concrete dam drain, measured monthly.

b. Two piezometers should be installed in the downstream slope of "full" earthfill dam section to measure phreatic (seepage) line, if any, monthly.

All of the foregoing are discussed in detail in the report.

It is possible that the probable maximum flood will never occur, but these dams, with the recommended changes and repairs, will fully provide full safety against such a happening.

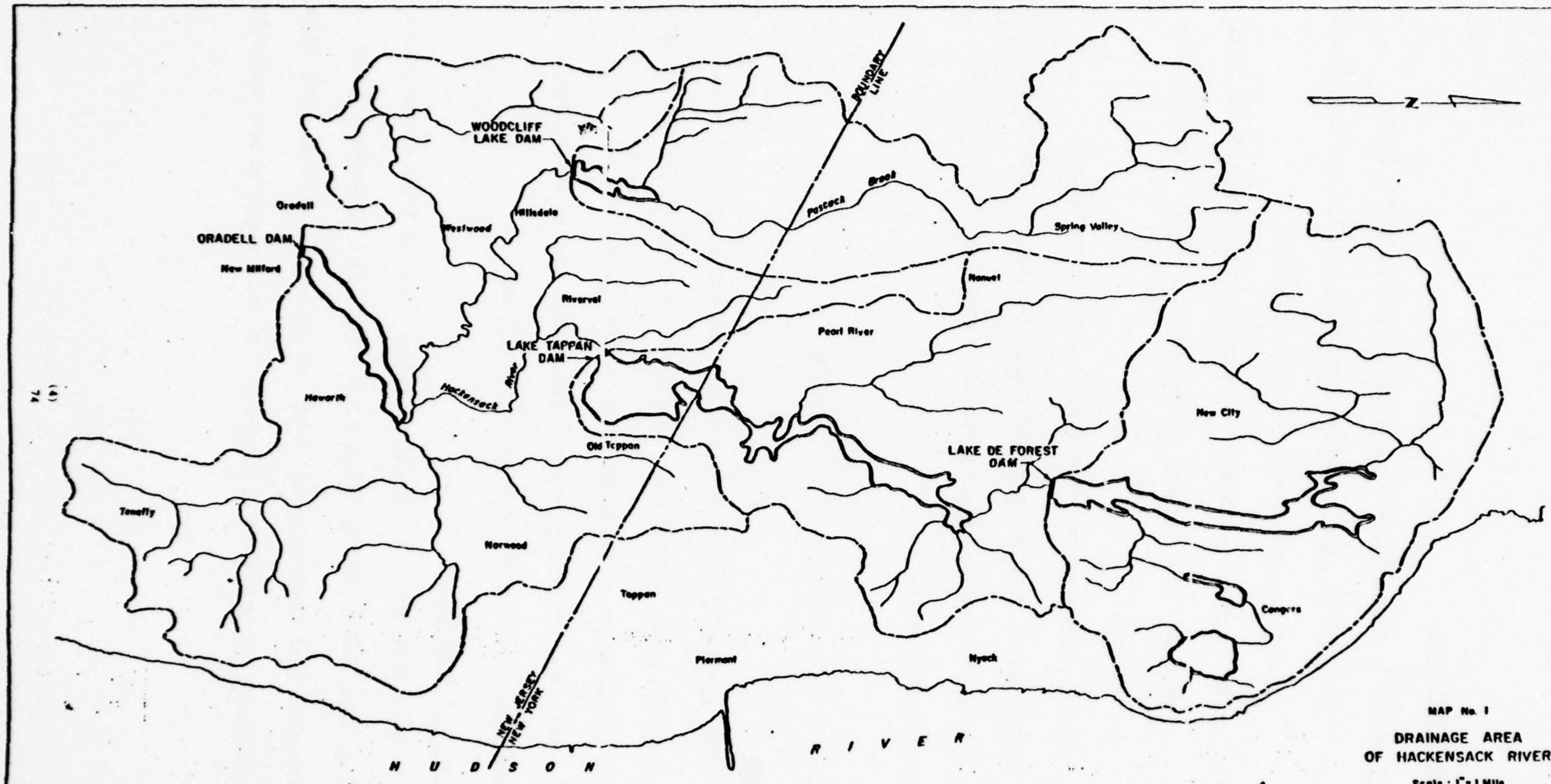
REPORT

Purpose

The purpose of this report is to analyze and examine in detail the Woodcliff Lake, Oradell, Lake Tappan, and Lake DeForest dams on the Hackensack River as to their safety under all conditions and to determine their possible defects, if any, and to recommend remedial changes and repairs as may be required. Locations of the dams are shown on the accompanying map, page 4.

Source of information

Basic data used in this report were obtained from construction drawings, reports, files, ASCE papers, USGS maps, road maps, geological folios and reports, hydrological and meterological data, seismological reports, and climatological reports supplemented by visual inspections of the structures and watersheds. Since



MAP No. 1
DRAINAGE AREA
OF HACKENSACK RIVER
Scale : 1" = 1 Mile

some of these projects were constructed early in this century, accurate data in some instances were scanty or difficult to obtain.

Hazardous aspect

The spillway of a storage dam must have a capability of passing a probable maximum flood without endangering the safety of the dam structure. The magnitude of the reservoir inflow flood to be used as a basis of design is related to the hazard potential and size of the dam and reservoir. In general, any dam located in an urban or built-up area is considered to be a high-hazard structure. Woodcliff Lake Dam, Oradell Dam, Lake Tappan Dam and Lake DeForest Dam are in this category. Although low in height above stream bed, the volumes of stored water impounded by these dams would normally classify the structures as of intermediate size. The spillway of such a structure is commonly designed to safely pass the flood which would be generated by the probable maximum precipitation which could occur over the watershed upstream from the dam. The frequency of occurrence of such a flood is not susceptible of determination.

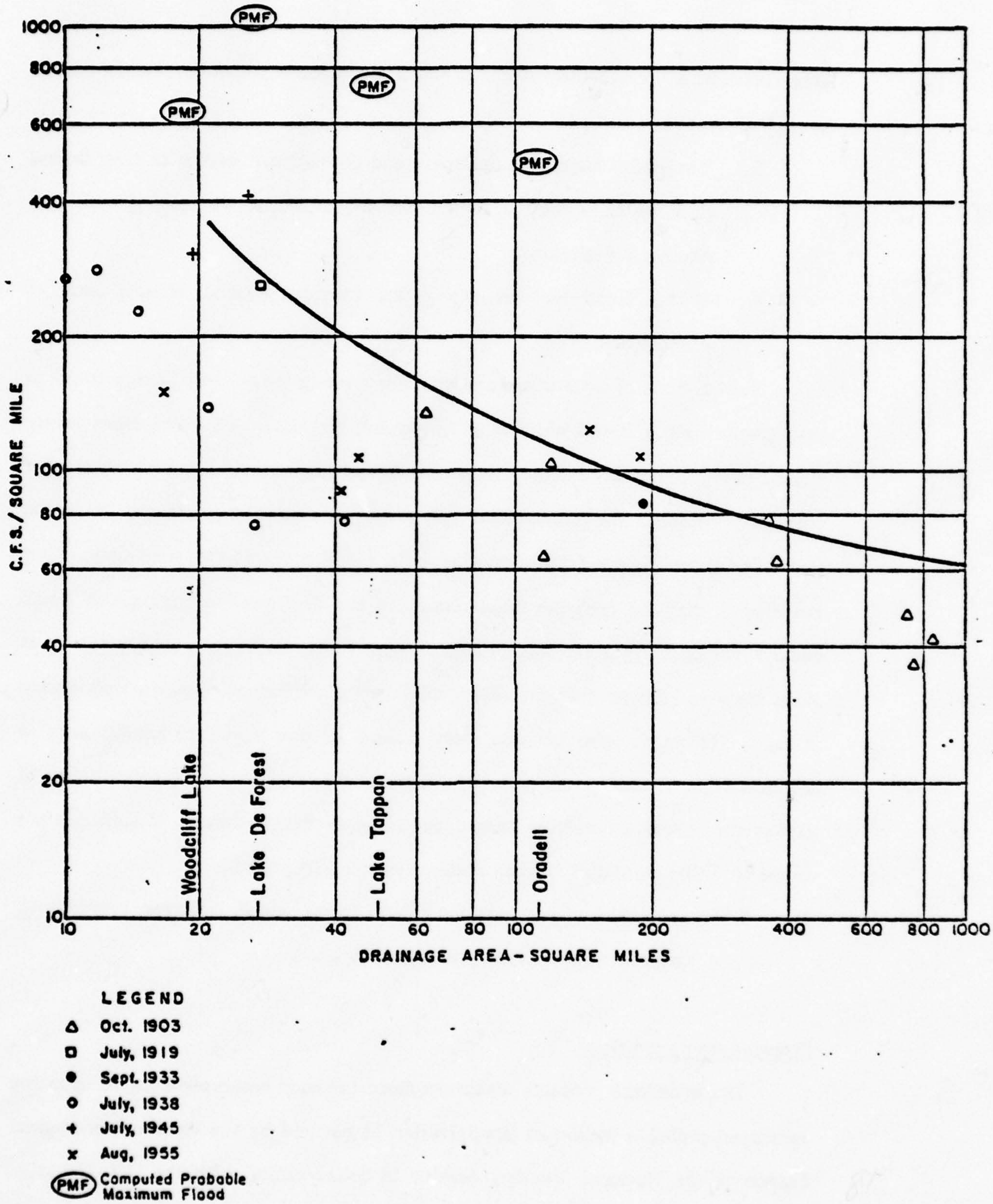
Method of dam safety analyses

The severe analyses for dam safety employed herein are in five steps, i.e.:

1. Determine the probable maximum flood hydrograph which could result from the probable maximum rainfall. This is the flood which may be expected from the most severe combination of critical meterologic and hydrologic conditions that are reasonably possible in the region.

2. Apply the hydrograph of the maximum flood to the reservoir being tested, assuming the reservoir is full normally at the start of the flood. Route the hydrograph through the reservoir taking into account the surcharge in the reservoir caused by the flood. This increase in temporary reservoir storage during the flood hydrograph is automatic in the case of ungated spillways and also in the case where the rated capacity of bascule gates is exceeded. This temporary storage greatly reduces the outflow peak. There are many ways to route floods through reservoirs and most of them are cut and try methods. The Horner method used in this report does not require cut-and-try but does require preparation of tables and graphs. Separate studies indicate that in the case of Tappan and Oradell reservoirs the outflows at the spillways could be reduced about 10% if the surcharge of other upstream reservoirs was taken into account. This further possible advantage of the surcharge effects of reservoirs upstream is not taken into account because this small advantage could possibly be offset by other factors such as the spillway gates of such dams being opened at the wrong time during the flood. Thus it is on the side of safety to consider each reservoir separately.

Ordinarily the floods of New Jersey streams are low in terms of cfs/sq.mi. To date conditions have not been right on the Hackensack for floods to approach the probable maximum but it can happen - especially at some time possibly even with a lesser rain on frozen ground. Furthermore, the drainages of these reservoirs are in the zone of hurricanes - so far, fortunately, only the edges of hurricanes have passed over the areas. The chart on page 7 shows unusual flood peaks which



Note:
Data From U.S.G.S.
Water Supply Paper No. 1672

UNUSUAL FLOOD PEAKS
NEW JERSEY

have occurred on New Jersey streams, and the computed probable maximum flood at each of the reservoirs.

3. Determine maximum upstream and downstream depths of flow (based upon the flood routing) at the dam and check for overtopping, stability, sliding, and stresses.

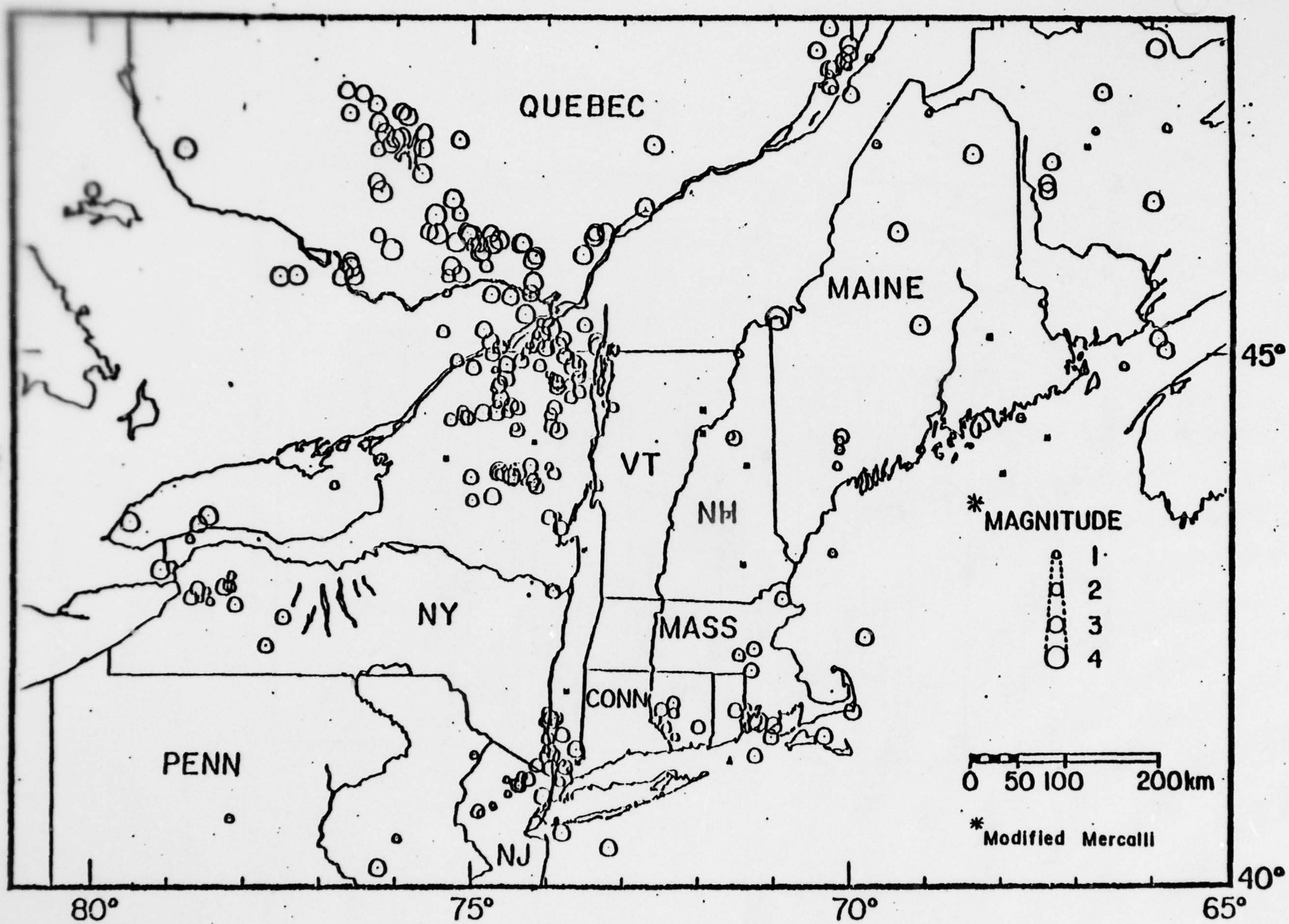
4. If the flood hazards are great, consider special effects such as earthquake.

The chances of an intensive earthquake shaking these dams is very small as they are located in Zone I which is of the lowest intensity. However, many minor quakes have occurrred. Furthermore, in compounding catastrophies which are not physically related, the frequencies of their respective occurrences must be multiplied to obtain the frequency when both would occur at the same time. Thus the chance that the probable maximum flood and an intensive earthquake would occur simultaneously is very highly improbable. However, if the probable maximum flood analyses indicate that the safety of a dam is questionable, then an allowance is made for earthquake because dam failure in this highly populated area is unthinkable. The map on page 9 indicates the locations and magnitudes of earthquake shocks which have been experienced in the northeast. Magnitudes are on the Modified Mercalli intensity scale as detailed on page 10.

5. Make field inspections and checks for seepages, leakages, movements, distortions, and rise in phreatic flow lines.

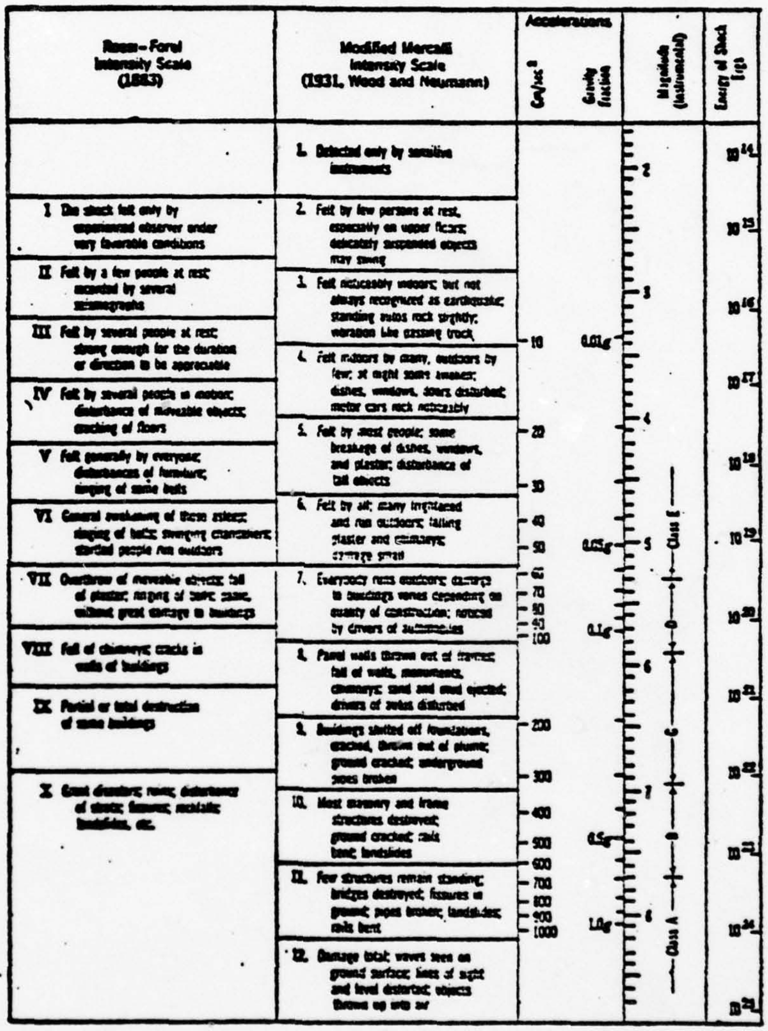
Probable maximum flood

The estimated probable maximum flood for each reservoir is based upon the estimated probable maximum precipitation as derived by the Hydrometerological Branch of the National Weather Service in collaboration with the U.S. Corps of



FROM "EARTHQUAKES, FAULTS AND NUCLEAR POWER PLANTS IN SOUTHERN NEW YORK-NORTHERN NEW JERSEY," AGGARIVAL AND SYKES, LAMONT-DOHERTY GEOLOGICAL OBSERVATORY

RICHTER



Approximate relationships, earthquake intensity, accelerations, and magnitude.

FROM AMERICAN CIVIL ENGINEERING
PRACTICE, VOLUME III, ABBETT

Engineers and is derived basically for six hours duration for ten square mile area, with curves and tables for corrections for other areas and durations.

The procedures for determining the probable maximum flood were developed by the Soil Conservation Service, Water Resources Council of the Bureau of Reclamation. There are two important factors affecting floods which are difficult to determine particularly in this instance. One item is the percentage of runoff and the other is the time of concentration.

The runoff percentage is rather low because the drainages are (1) gently rolling or relatively flat except at the basin rim, (2) the drainage areas were at one time glaciated leaving large areas of sand, gravel, swamps, and bogs, (3) much of the area is covered by roads, buildings, and improvements, and (4) there are very extensive forest, humus, and brush areas as well as some cultivated areas.

The time of concentration is relatively slow because the drainages are rolling or relatively flat except near the basin rims. Furthermore during large floods, the flow would spread out into the forests, fields, swamps, brush, sand and gravel areas and pits, and urban areas all of which would retard the flow. Much of the flow path would be through reservoirs. After review and study, an average velocity of two feet per second was adopted for the greatly widened and roughened channels and six feet per second through the relatively long reservoirs.

Description of the dams and reservoirs

All of the dams and reservoirs analyzed herein are used exclusively for water supply and all were constructed on swampy areas which had no other use. The dams are low in height but the reservoirs are long and the storages are large. The Oradell and Tappan dams were constructed on very poor foundations which required highly unusual and unorthodox designs.

Woodcliff Lake dam, also known as Hillsdale dam, was constructed in 1903-05 on Pascack creek, a tributary of the Hackensack River, and is of the homogeneous earthfill type having a concrete core wall and steel sheet piling. It appears that the earthfill was not definitely compacted. The slopes of this fill are very steep, two horizontal to one vertical, and the freeboard is very scanty. It has an ungated concrete spillway and a concrete spillway chute. Flashboards have been employed at times; these have been discontinued. Extensive repairs and modification were made to the spillway and chute in recent years. The spillway capacity is much less than would be dictated by modern engineering practice.

Oradell dam, the lowermost dam on the Hackensack River, was constructed in 1923, and is an unorthodox type consisting of an Ambursen overflow type for the ungated spillway section but is constructed in reverse. The inside of the dam was filled hydraulically with clay. Due to the poor foundations, the buttresses rest on many wooden piling. A steel sheet piling cutoff is provided. The nonoverflow section of the dam consists of seven 7' x 9' slide type sluice gates located between concrete buttresses. This dam was repaired in recent years. The dam was resurfaced, the clay filling was grouted, and the sluice gates were reconditioned and repaired. A portable electric operator was provided to facilitate sluice gate operation.

Lake Tappan dam was constructed in 1964 and is an unorthodox, special articulated, flexible concrete type dam "floating" on a foundation of sand. It has four 6' x 50' automatic bascule type spillway gates and has homogeneous earthfill type ends. Leakage is controlled by slabs, aprons, and partial sheet piling cutoffs.

DeForest dam, the uppermost dam on the Hackensack River, was constructed in 1954, and consists of two 5' x 50' automatic bascule type spillway gates located on a concrete overflow gravity section which connects to two nonoverflow concrete gravity dams each about 150 feet in length. The right bank nonoverflow gravity dam connects to a long homogeneous earthfill dam. The concrete sections are situated on rock while the earthfill is partially connected to rock by steel sheet piling. This dam has a large freeboard.

Data concerning these four dams and reservoirs are summarized in the tabulation on page 14. Details and dimensions of Lake DeForest dam, Lake Tappan dam and Oradell dam and our findings with regard to their safety under probable maximum flow conditions are presented in the following pages. Studies of Woodcliff Lake dam are still in progress; a separate report of our findings regarding the safety of that sturcture and our recommendations for modifications to the existing spillway will be submitted shortly.

	Project			
	Woodcliff	Oradell	Tappan	DeForest
Construction date	1903-05	1923	1966	1954
Drainage area (sq.mi.)	18.4	93.6+18.4	49.4	26.75
Reservoir volume (acre-feet)	2728	8838	10,894	17,185
Reservoir area (acres)	171	670	1,255	1,016
Reservoir length (miles)	1.5	3.0	6.0	4.75
Elevation spillway crest	94.33	23.16	49.0	80.0
Normal W.S. elevation	94.33	23.16	55.0	85.0
Normal maximum W.S. elevation	97.4	25.67	55.0	85.0
Elevation top of dam	100.0	25.67	64.0	100.0
Elevation bottom of dam	65.0	2.67	30.0	46
Maximum height of dam	35	23.0	34	54
Spillway	82'-no gates	(331'-no gates (7-7'x9' sluices	4-6'x50' bascule gates	2-5'x50' bascule gates
Spillway capacity (cfs)	2,200 at Elev. 99	13,000 at Elev. 25.67	10,000 at Elev. 55	10,000 at Elev. 90
Spillway capacity (cfs/sq.mi.)	121	116	202	400
Probable max. inflow peak (cfs)	11,400	49,500	38,000	27,800
Probable max. inflow peak (cfs/sq.mi.)	619.6	437.5	769	1,039
Probable max. outflow (cfs)	9,800	(26,500 crest (9250 sluice	27,000	14,000
Max. reservoir elevation during P.M.F.	Over 100.0	28.70	59.8	92.2

ORADELL DAM

Oradell Dam was constructed in 1923. It is an unorthodox low dam located on a very poor foundation but, and aside from maintenance repairs, has given no trouble over the years. It consists of an Ambursen type dam built in reverse and filled hydraulically with clayey material. It is supported on a forest of timber piles and cutoff is accomplished by a steel sheet piling. The ungated spillway is 331 feet in length and is designed to operate normally under one foot of head but up to three feet in emergencies. Most of the floods are carried by seven electrically operated 7' x 9' slide gate sluices adjacent to the dam. The discharge capacity of the sluiceway, with gates fully open and water at spillway crest level is approximately 8,000 cubic feet per second.

In recent years the following maintenance to the dam was done:

1. The interior was grouted with 740 sacks of cement, sand, and bentonite grout to replace lost clay.
2. A slab one foot thick was placed on the downstream face (six inches thick on the crest).
3. The sluice gates were repaired and reconditioned.

The drainage area is 112 square miles of which 93.6 square miles are on the Hackensack River and 18.4 square miles are on the tributary Pascack Brook. The reservoir area and volume are 670 acres and 8,838 acre feet, respectively, at spillway crest elevation.

A plan view of the dam and generalized sections of the dam and sluiceway are shown on pages 30, 31, and 32.

Discharge through the sluice gates at Oradell Dam is adjusted to suit the raw water requirements of the New Milford filtration plant located a short way downstream. In addition, the routine duties of the Watershed Patrol Forces require that this dam, as well as the other three dams in the watershed, be visited at two-hour intervals throughout the day and night. Water surface elevations are measured and gate operating positions are logged at those visits. When rainfall occurs, in excess of one-half inch, each of the dams is visited every hour until rainfall ceases and the runoff crest has passed. At Oradell Dam the gates are progressively opened to pass flow through the sluiceway, to maintain the reservoir level at spillway crest elevation. With the sluice gates fully opened, and the reservoir level at the sluiceway top slab elevation 25.67, total discharge over the spillway and through the gates is 13,000 cfs.

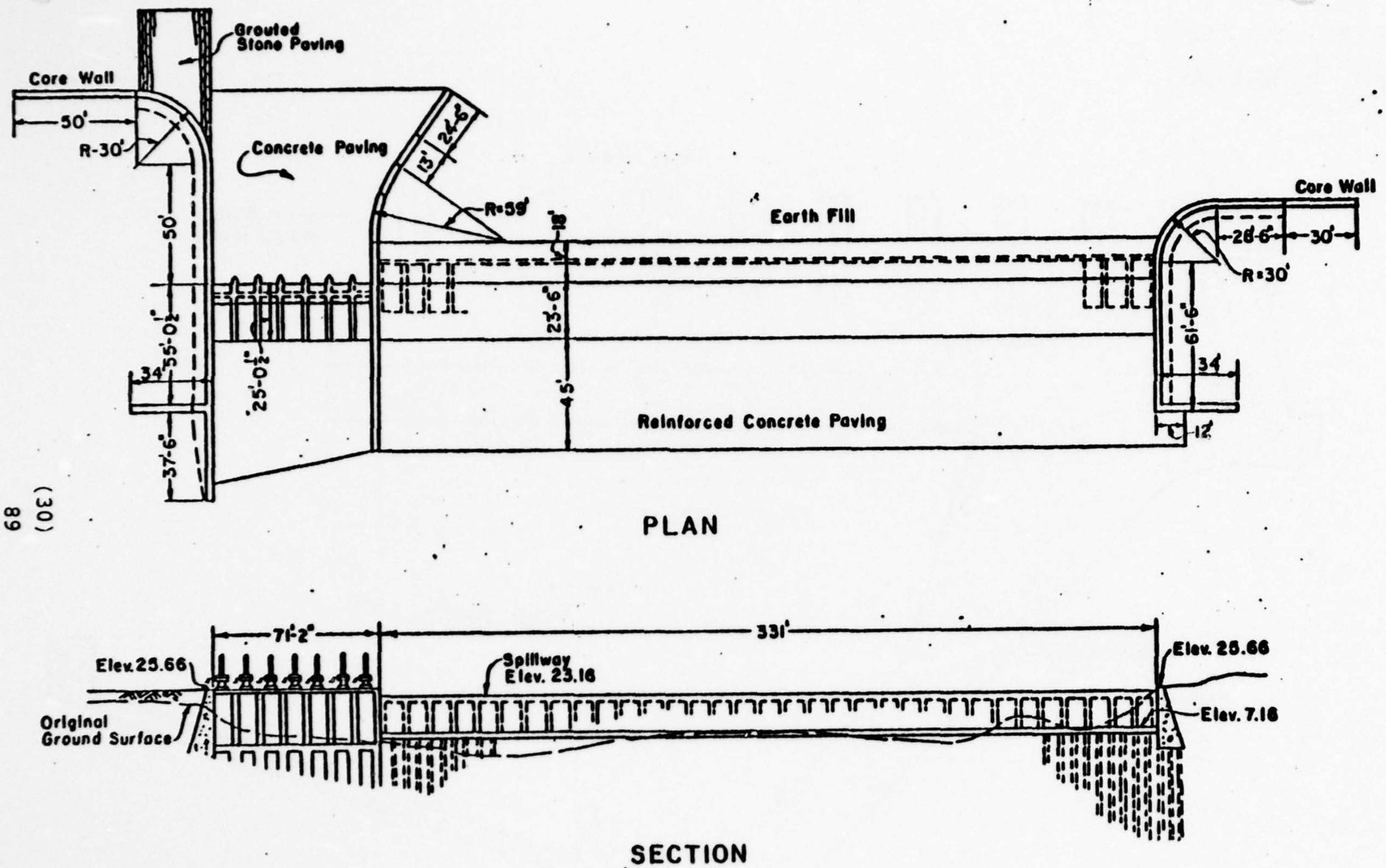
The reservoir inflow flood peak under probable maximum flood-producing conditions is estimated to be 50,000 cfs, equivalent to 446 cfs per square mile from the 112 square mile drainage area. A peak flow of this magnitude would be produced by 17.05 inches of rainfall occurring in 6 hours. Computations of the probable maximum precipitation and the corresponding probable maximum flood for the watershed tributary to Oradell Dam are presented in Appendix A. The reservoir inflow hydrograph was computed first for the entire drainage area and then by combining separate computations of flow from Pascack Brook with that from the remainder of the watershed. The latter method produced a slightly higher peak inflow and was selected for routing through the reservoir. Reservoir inflow and outflow hydrographs for the probable maximum flood are shown on page 33. The peak outflow over the dam would be 28,500 cfs if 7,000 cfs passes through the

sluices. This would produce a water depth of 5.74 feet on the crest of the spillway. The corresponding differential head of 14.9 feet on the sluice gates would cause a flow of 9,500 cfs through the gates instead of 7,000 as assumed. Hence it is necessary to cut and try to obtain a balance in flows. Figures that satisfy this are 26,250 cfs over the dam and 9,250 cfs through the sluices with the tailwater at elevation 14.0. The head on the dam crest would be 5.54 feet, equivalent to a reservoir water surface at elevation 28.70. At that elevation flow would be passing over the sluice gate structure and over the adjacent lands and railroad.

Stability analyses of the spillway section and of the sluiceway were made for peak outflow conditions. Details of the computations and the resultant forces and their locations are presented in Appendix B. Stability of the spillway section is excellent as the resultant falls within the middle third of the base. Sliding resistance is ample without the shear resistance of the piling. Stresses are low and are positive, with the load on the foundation assumed to be carried entirely by the piers. Sliding resistance of the sluiceway is satisfactory when passive resistance of the backfill is taken into account. Stresses are low. A small amount of tension exists on the highest pier. However, this pier adjoins the spillway section which has piling. The resultant falls slightly in the downstream third of the base which indicates a small amount of tension but not overturning. The position of the resultant is more favorable as the piers reduce in height.

The condition of the structure was examined in the field. No evidence of unsafe conditions or inadequate maintenance was noted. The sluiceway as well as the surrounding lands and railroad is bound to be flooded in major floods since there is no way to increase the capacity of the spillway. In major floods power might fail

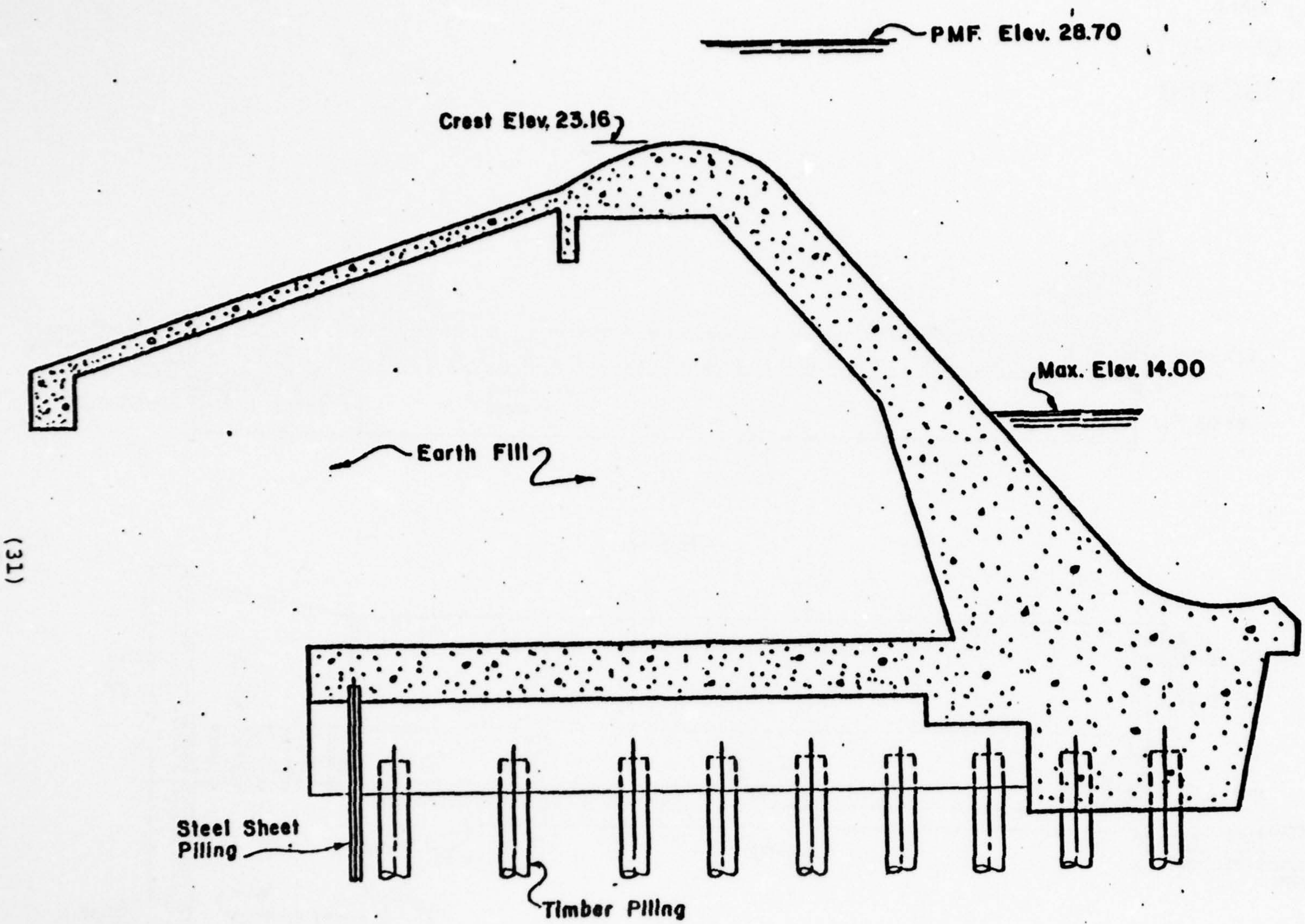
and the sluice gates, which have considerable capacity, could not be operated. We recommend that an on-site auxiliary power plant be installed for emergency use. Or the sluice gates could be converted to hydraulic operation so that they could operate inundated.



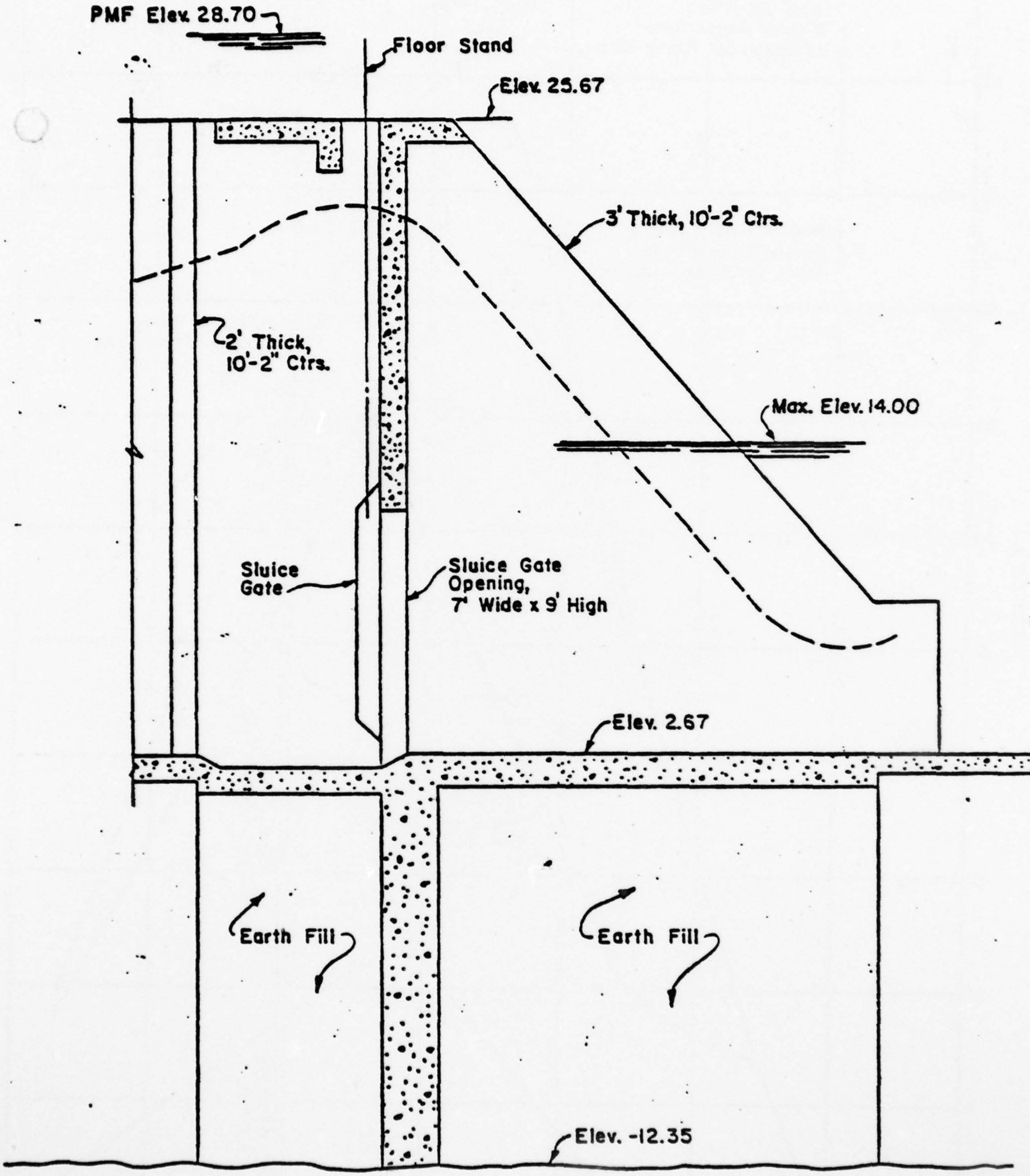
ORADELL DAM

PLAN AND SECTION

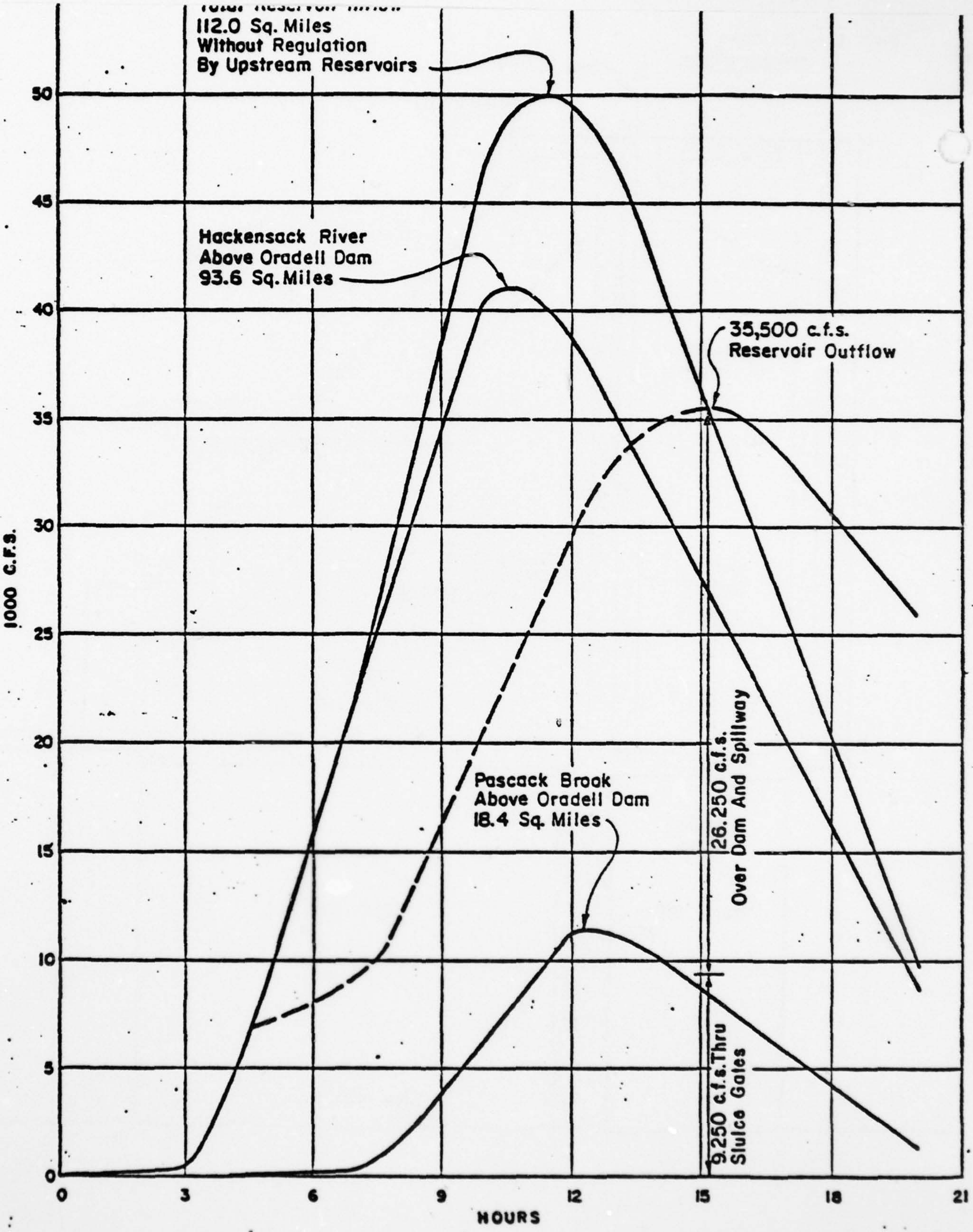
Scale 1" = 60'



ORADELL DAM
SPILLWAY SECTION
Scal "= 5'



ORADELL DAM
SLUICEWAY SECTION
Scale: 1" = 5'



(33)
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ORADELL DAM

PROBABLE MAXIMUM FLOOD

INFLOW AND OUTFLOW HYDROGRAPHS

AD-A060 025 BAKER (MICHAEL) JR INC BEAVER PA F/G 13/2
NATIONAL DAM SAFETY PROGRAM. ORADELL RESERVOIR DAM (NJ00258), H--ETC(U)
AUG 78 M BAKER DACW61-78-C-0141
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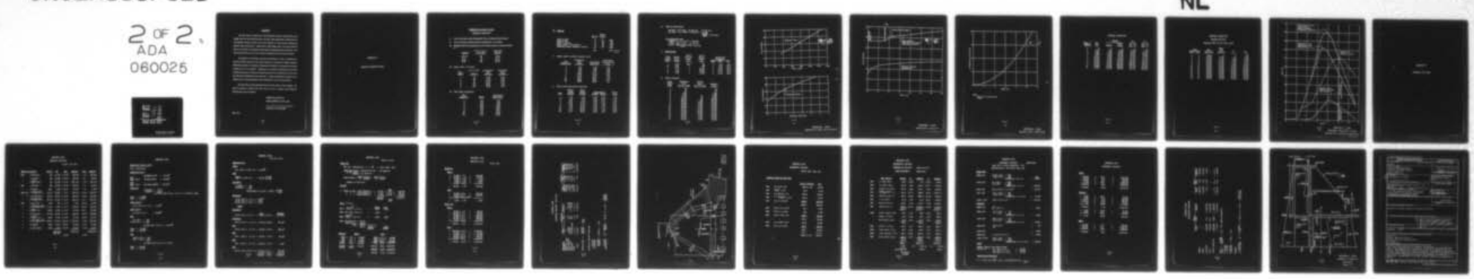

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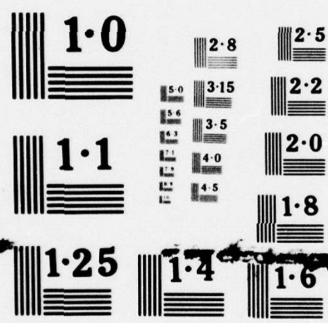
1·0
2·8
2·5
5·0
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3·15
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1·25
1·4
1·6

CLOSURE

We have made an inspection of the Woodcliff Lake Dam, Oradell Dam, Lake Tappan Dam and Lake DeForest Dam, and have made independent computations of the probable maximum floods and of the stability of the structures during such extreme flood occurrences. Oradell Dam, Lake Tappan Dam, and Lake DeForest Dam in our opinion are completely safe under all foreseeable flood conditions. This report presents the results of our studies and findings for these three structures.

The spillway of Woodcliff Lake Dam, constructed in 1903, is inadequate to pass the probable maximum flood peak computed to present-day design standards. Preliminary layouts and designs of alternative methods of increasing the spillway capacity are now in progress. Foundation and material investigations have been completed; laboratory testing of material sampled from the spillway area and from the main dam embankment is now underway. A report presenting the results of our studies and our recommendations will be submitted to you in the near future.

We trust that the data presented herein will be useful to the Company. We shall be pleased to discuss the data with you and to furnish such additional information as may be desired.

Respectfully submitted

BUCK, SEIFERT & JOST, INC.

Lawrence A. Leyenberger

Lawrence A. Leyenberger

July, 1978

APPENDIX A

SPILLWAY DESIGN FLOODS

PROBABLE MAXIMUM FLOOD

ORADELL RESERVOIR

1. Area = 93.6 square miles (Hackensack River, excluding Pascack Brook)

2. 6 hr, 10 sq.mile, probable maximum precipitation = 24.5 inches

3a. Probable maximum precipitation for 93.6 square miles for various durations is:

Duration	% of 10 sq.mi. 6 hr. value	Total PMP (inches)
0-6 hrs	80	19.60
0-12	87	21.31
0-24	95	23.28
0-48	108	26.46

3b. Hourly PMP, 6 hr period

Time (hours)	% 6 hr PMP (inches)	Accumulated PMP (inches)	Incremental PMP (inches)
1	49	9.60	9.60
2	64	12.54	2.94
3	75	14.70	2.16
4	84	16.46	1.76
5	92	18.03	1.57
6	100	19.60	1.57

3c. PMP design arrangement

Time (ending hr.)	Inches	Accumulated PMP (inches)
1	1.57	1.57
2	1.76	3.33
3	2.16	5.49
4	9.60	15.09
5	2.94	18.03
6	1.57	19.60
12	1.71	21.31
24	1.97	23.28
48	2.18	26.46

3d. Land use

	Est. %	Runoff Curve		
Trees & brush	57	25	1425	
Grass & veg.	6	30	180	
Farm & orchards	6	40	240	
Roads, bldgs., developments	25	70	1750	
Reservoirs, lakes, swamps & streams	6	100	600	
	100		4195	41.95

4. Design rainfall (probable maximum flood)

Time (Ending Hour)	Accumulated PMP (inches)	Accumulated PMP x 87%	Incremental Design Rainfall MPM
1	1.57	1.36	1.36
2	3.33	2.90	1.54
3	5.49	4.78	1.88
4	15.09	13.13	8.35
5	18.03	15.69	2.56
6	19.60	17.05	1.36
12	21.31	18.54	1.49
24	23.28	20.25	1.71
48	26.46	23.02	2.77

5. Direct runoff increments for PMF flood

Time	Increm. Design Rainfall	Accum. Design Rainfall	Direct Runoff Accum.	Direct Runoff Increm.	Increm. Loss
1	1.36	1.36	0.03	0.03	1.33
2	1.54	2.90	0.05	0.02	1.52
3	1.88	4.78	0.26	0.21	1.67
4	8.35	13.13	4.45	4.19	4.16
5	2.56	15.69	6.25	1.80	0.76
6	1.36	17.05	7.27	1.02	0.34
12	1.49	18.54	8.42	1.15	0.34
24	1.71	20.25	9.78	1.36	0.35
48	2.77	23.02	12.05	2.27	0.50

6. Time of concentration

On land = 10.3 miles = 54,384 ft/2 = 27,192 sec.
On water = 13.6 miles = 71,808 ft/6 = 11,968

$$\frac{39,160}{3600} = 10.9 \text{ hours}$$

For one hour increment

T peak = ½ + 0.6 x 10.9 = 7.04 hrs
T base = 2.67 x 7.04 = 18.8 hrs

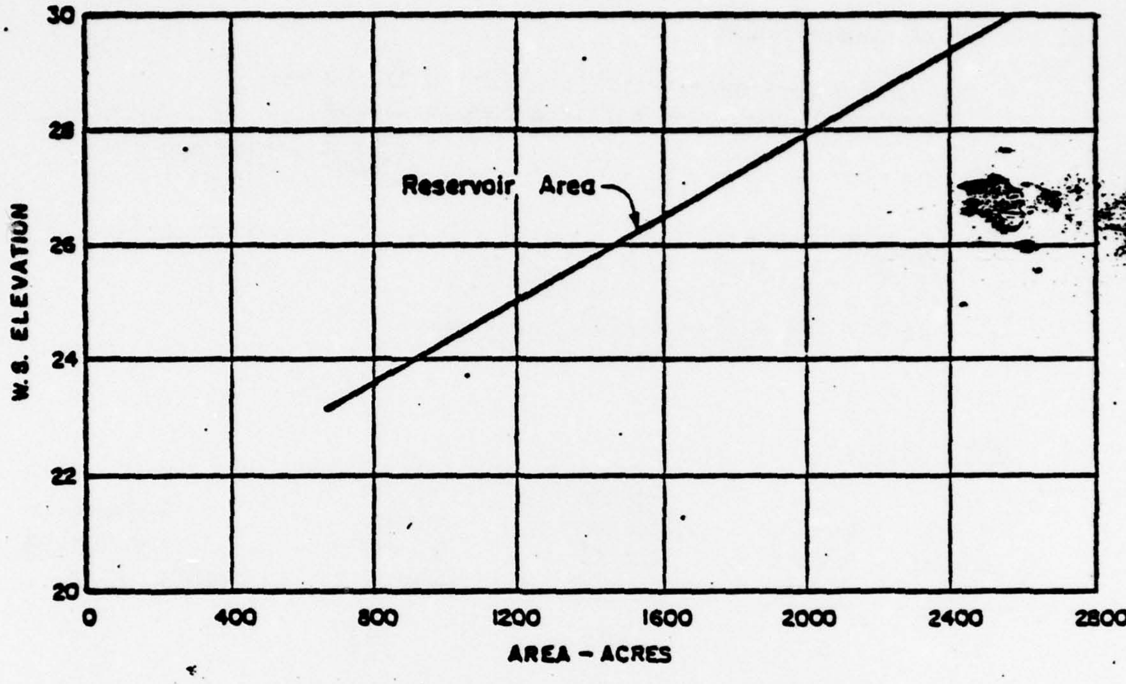
$$q\ \text{peak} = \frac{484 \times 93.6 \times 1.00}{7.04} = 6435 \text{ cfs}$$

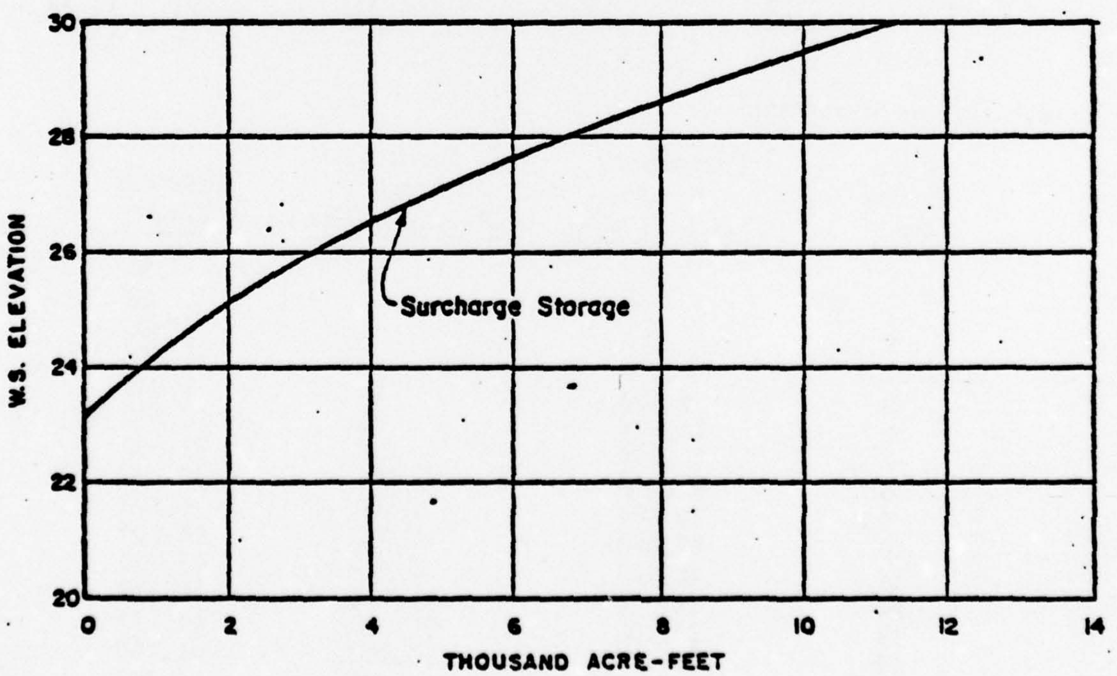
7. Plotting table

Time (ending hour)	Increm. Runoff inches	qp for 1.00 inch Runoff cfs	qp for Increm. Runoff cfs	Incremental Hydrograph		
				Begin	Peak	End
1	0.03	6435	193	0	7.04	18.8
2	0.02	"	129	1	8.04	19.8
3	0.21	"	1,351	2	9.04	20.8
4	4.19	"	26,963	3	10.04	21.8
5	1.80	"	11,583	4	11.04	22.8
6	1.02	"	6,564	5	12.04	23.8

8. Inflow hydrograph

Time Ending Hour	Hackensack River 93.6 sq. miles	Pascack Brook 18.4 sq. miles	Reservoir Inflow
1	30		30
2	70		70
3	300		300
4	4,200		4,200
5	9,400	20	9,400
6	15,900	50	16,000
7	22,200	100	22,300
8	28,400	1,700	30,100
9	34,600	3,800	38,400
10	40,500	6,200	46,700
11	40,900	8,700	49,600
12	38,700	11,100	49,800
13	35,100	11,000	46,100
14	31,200	10,200	41,400
15	27,600	8,700	36,300
16	23,700	7,200	30,900
17	20,000	5,700	25,700
18	16,100	4,200	20,300





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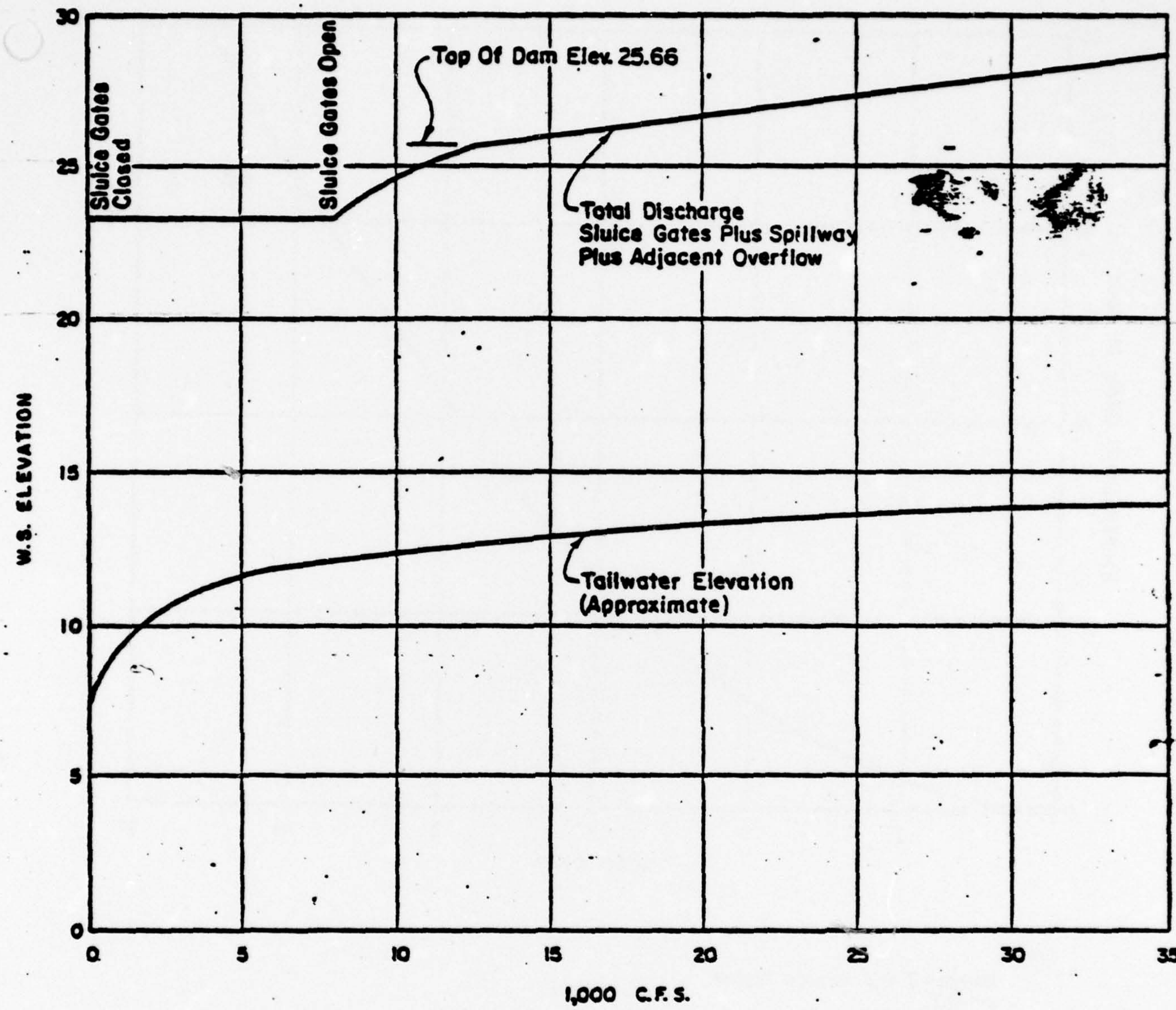
ORADELL DAM
RESERVOIR AREA AND STORAGE



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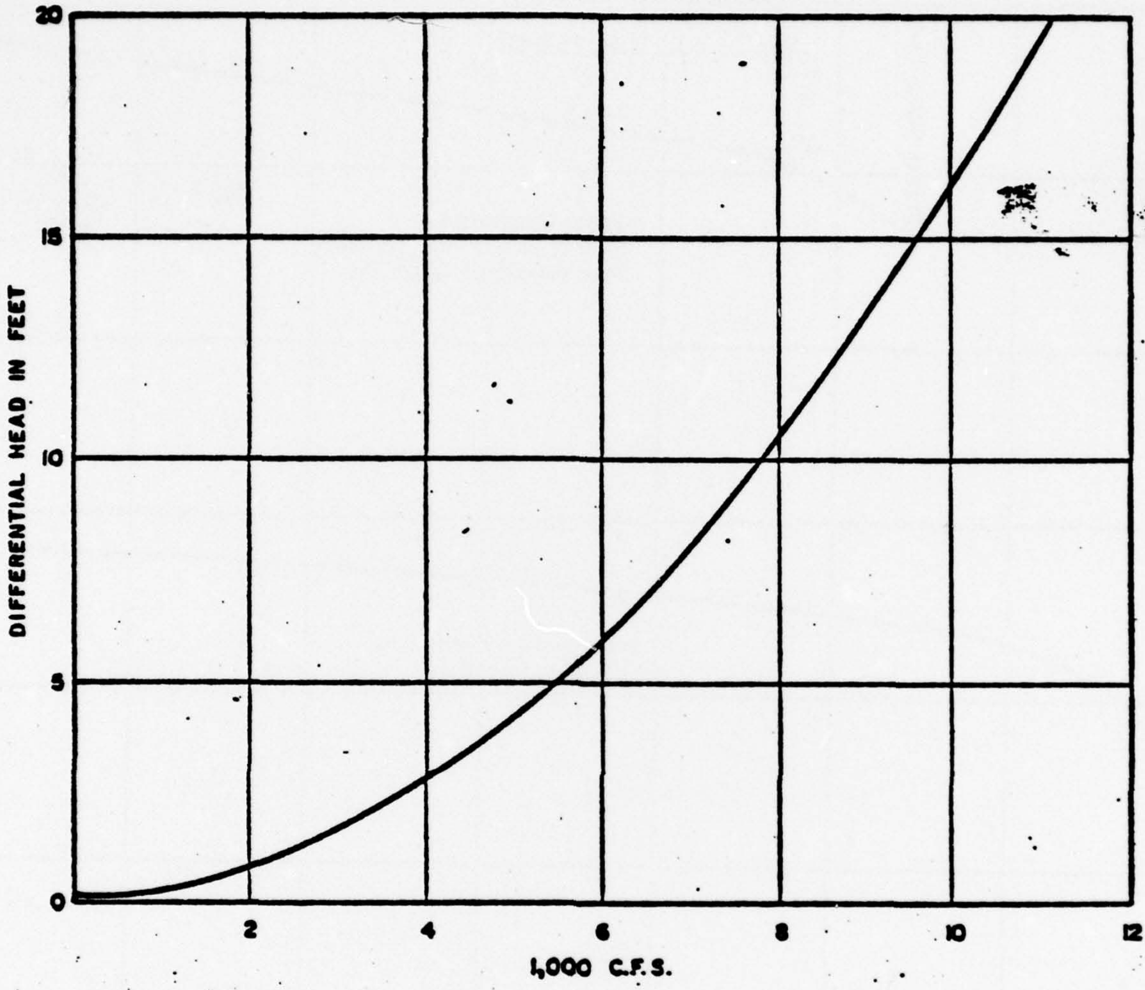
ORADELL DAM
DISCHARGE CAPACITY



Note:
Seven - 7' x 9' Sluice Gates
c = 0.7

ORADELL DAM
SLUICE GATE DISCHARGE

ORADELL RESERVOIR

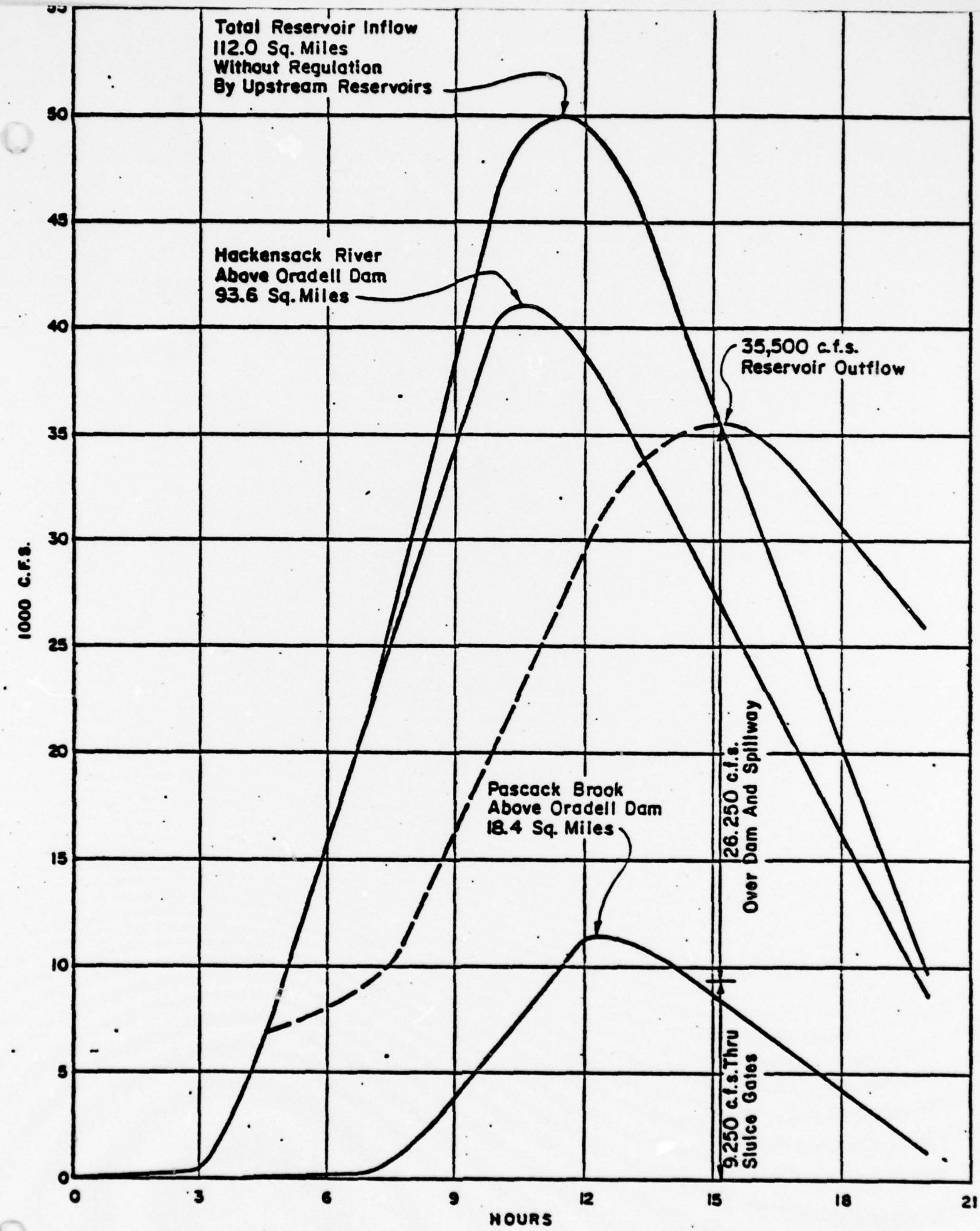
Depth on Spillway	Spill cfs	Storage Ac.Ft.	Storage Sec.ft. hours	F_2 $\frac{S}{\Delta T}+\frac{O}{2}$	F_1 $\frac{S}{\Delta T}-\frac{O}{2}$
0	0	0	0	0	0
1	1,160	700	8,400	8,980	7,820
2	3,281	1,900	22,800	23,240	21,160
3	6,028	3,000	36,000	39,014	32,986
4	9,501	4,600	55,200	59,950	50,450
5	13,647	6,500	78,000	84,823	71,177
6	18,296	8,500	102,000	111,148	92,852
7	25,648	11,000	133,000	144,824	119,176
8	31,662	13,900	166,800	182,631	150,964
9	38,500	17,000	204,000	223,250	184,750
10	44,268	20,000	240,000	262,134	217,866
11	51,072	24,000	288,000	313,536	262,464

CRADELL RESERVOIR

FLOOD ROUTING

Assuming 7000 cfs Thru Sluice Gates

Time Hour	to Hour	Mean Inflow (cfs)	Inflow Less 7000 cfs	0_1	0_2
4	5	6,800	0	0	100
5	6	12,700	5,700	100	1,000
6	7	19,200	12,200	1,000	2,200
7	8	26,200	19,200	2,200	5,400
8	9	34,200	27,200	5,400	9,200
9	10	42,600	35,600	9,200	13,600
10	11	48,200	41,200	13,600	18,250
11	12	49,700	42,700	18,250	22,500
12	13	48,000	41,000	22,500	26,000
13	14	43,800	36,800	26,000	27,800
14	15	38,800	31,800	27,800	28,500
15	16	33,600	26,600	28,500	28,100
16	17	28,300	21,300	28,100	25,900
17	18	23,000	16,000	25,900	23,300



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ORADELL DAM
PROBABLE MAXIMUM FLOOD
INFLOW AND OUTFLOW HYDROGRAPHS

APPENDIX B

STABILITY ANALYSES

ORADELL DAM

SPILLWAY SECTION

(1 unit = 10' ctrs)

Weight of Concrete		Cu.Ft.	Wt.	Hor.	Moment	Vert.	Moment
Slab	= 20.5x10x2 =	410	61,500	x 10.67'	656,205	4.0	246,000
"	= 3.6x10x3 =	108	16,200	x 22.25	360,450	3.5	56,700
"	= $\frac{6.5+8}{2}$x10x6 =	435	65,250	x 27.92	1,821,780	3.16	206,190
Pier	= 20.5x4x3 =	246	36,900	x 10.38	383,022	1.5	55,350
"	= 3.6x4x2 =	28.8	4,320	x 22.33	96,466	1.0	4,320
Main pier	= $\frac{9.33x16.35x2}{2}$ =	152.55	22,882	x 8.75	200,218	10.33	236,371
" "	= 16.35x4.88x2 =	159.58	23,936	x 12.25	293,216	13.16	314,998
" "	= $\frac{16.35x14.5x2}{2}$ =	237.08	35,562	x 19.42	690,614	10.5	373,401
Face	= 0.5x1.5x8 =	6.00	900	x 8.75	7,875	18.25	16,425
"	= $\frac{1.58+2.6x3.67x8}{2}$ =	61.36	9,204	x 11.13	102,440	20.16	185,553
"	= $\frac{2.6x2}{2}$ x 8 =	20.80	3,120	x 14.25	44,460	20.5	63,960
"	= 8.75x2.5x8 =	175.00	26,250	x 17.16	450,450	16.75	439,688
"	= $\frac{2.5x2.3}{2}$ x 8 =	23.00	3,450	x 21.00	72,450	13.25	45,713
"	= $\frac{3.5+7.58x11.5x8}{2}$ =	509.68	76,452	x 23.50	1,796,622	8.38	640,668
"	= 0.67x3x8 =	16.08	2,412	x 30.25	72,963	5.25	12,663
"	= 1.25x1.25x8 =	12.50	1.875	x 32.25	60,469	5.16	9,675
			390,213# (for 10')		7,109,700		2,907,675
				18.22'		7.45'	

ORADELL DAM

Water load (10'ctr to ctr)

W.S. = 28.70 elev.

Lower pier (hor.)

	25.58x62.4.3'x4'	=	19,154#
Floor Slab (hor.)	23.0x62.4x8'x2'	=	22,963#
Floor Slab (vert.)	22.16x62.4x8x21½	=	237,839#

Upper pier

22.04x62.4 = 1375.3
5.54x62.4 = 345.7
2) 1721.0 (860.5 ave. x 19.0 x 2 = 32,699 on slope

Horiz. = 27,800#
Vert. = 17,000#

Crest (horiz.)

6.75 x 62.4 x 8 x 2.5 = 8,424#

Crest (vert.)

8 x 62.4 x 8 x 5 = 19,968#

Slab

8 x 62.4 = 499
14.33 x 62.4 = 894

2) 1393 (697 x 8.6 x 8 = 48,160#

Horiz. = 35,000#
Vert. = 33,000#

Slab

14.33 x 62.4 = 894
22.0 x 62.4 = 1372

2) 2266 (1133 x 8.0' x 8 = 75,512

Horiz. = 68,500
Vert. = 22,600

ORADELL DAM

(10' ctr to ctr)

Backwater load

Apron

6.5 x 62.4 x 3.25 x 10 = 13,182#

Slab

$\frac{7x62.4}{2}$ x 10.33 x 10 = 22,561 (15,000 (17,500

Foundation

6.25x62.4 = 390
13.25x62.4 = 827

2)1217(608.5 x 10 x 7.00 = 42,595 = (42,000 (9,000

Foundation

9.75 x 62.4 x 6 x 1 = 3,650#
11.25 x 62.4 x 6 x 2 = 8,424
12.75 x 62.4 x 10 x 1 = 7,956

Uplift

Foundation

	Lbs		Moment
13.33 x 62.4 x 10 x 7 =	58,225	x 27.67 =	1,611,098

Foundation

10.33 x 62.4 x 6 x 3.5 =	13,536	x 22.25 =	301,187

Slab

10.33 x 62.4 x 6 x 19 =	73,483	x 11.00 =	808,310

Slab

24.0 x 62.4 x 6 x 1½ =	13,478	x 0.75 =	10,108

Slab

27.0 x 62.4 x 4 x 1½ =	10,108	x 0.75 =	7,581

Slab

12.33 x 62.4 x 22.5 x 4 =	69,245	x 13.0 =	900,188
=	13,000	x 30.75 =	399,750
	251,075	16.08	4,038,222

ORADELL DAM

(10' ctr to ctr)

Earth load

Wt. dry = 100 lbs/cu.ft. ϕ = 30° = 2.67, voids = 40%

Submerged weight = 100-62.4/(1-0.40) = 63 lbs/cu.ft.

or $100 \frac{(2.67-1)}{(2.67)}$ = 63 lbs/cu.ft.

$$\text{Earth pressure} = \frac{63xh^2}{2}\,\frac{(1-\sin\phi)}{(1+\sin\phi)} = \frac{63xh^2}{2}\,\frac{(1-0.5)}{(1+0.5)}$$

acting as a water load.

Vertical

				arm		
1. Weight on slab = 63x14.00x8x13.5'	=	95,256	x	6.75	=	642,978
" " " = 63x10.90x8x5.67	=	31,149	x	17.00	=	529,533
" " " = 63x3.75x8x2.33	=	4,404	x	20.40	=	89,842
		130,809#		9.65		1,262,353

Horiz. (& vert.)

		Horiz.	Vert.
Pier = $\frac{63 \times \overline{16}^2}{2} \times 1/3 \times 2$	=	5,376	3,200
Pier = 63 x 1/3 x 17.5 x 4	=	1,470	-
Slab = $\frac{63x\overline{6.5}^2}{2} \times 1/3 \times 8$	=	3,549	4,300

Slab = 63x1/3x14.16 = 297.36
63x1/3x 6.5 = 136.50
7.66 = 216.93 x 7.66 x 8 =
(ave.) 13,293 3,200
23,688 10,700

Moments		Horiz.				Vert.				
5376	x	8.5	=	45,659	3200	x	2.5	=	+8,000
1470		1.5	=	5,205	4300	x	17.25	=	-74,175
3549		14.9	=	52,880	3200	x	20.5	=	-65,600
13,293		8.4	=	111,661	4300		30.65		-131,775
23,688		8.99		212,441					
					130,809		9.65	+	1,262,353
					126,509		8.94	+	1,130,578

ORADELL DAM

SPILLWAY DAM

10 ft. ctrs.

Backwater

Horiz.

+	17,500	x	7.6	=	+133,000
+	42,000	x	1.75	=	+ 73,500
-	3,650	x	2.50	=	- 9,125
-	8,424	x	1.00	=	- 8,424
-	7,950	x	-0.50	=	+ 3,975
+	39,476		4.89		+192,926

Vert.

+	15,000	x	27.0	=	+405,000
-	13,182	x	30.5	=	-402,051
-	9,000		(inc.in uplift)		-
+	1,818		1.62		+ 2,949

Water load

Horiz.

-	19,154	x	1.5	=	- 28,731
-	22,963	x	4.0	=	- 91,852
-	27,800	x	11.8	=	-328,040
-	35,000	x	15.6	=	-546,000
-	68,500	x	8.5	=	-582,250
-	8,424	x	20.33	=	-171,260
-	181,841		9.61		-1,748,133

Vert.

+	237,839	x	12.75	=	+3,032,447
+	17,000	x	4.4	=	+ 74,800
-	19,968	x	11.0	=	- 219,648
-	33,000	x	16.65	=	- 549,450
-	22,600	x	20.5	=	- 463,300
+	179,271		10.46	=	+1,874,849

ORADELL DAM

SPILLWAY SECTION (10' ctrs.)

Description	Horizontal Load	Horizontal Arm	Horizontal Moment	Vertical Load	Vertical Arm	Vertical Moment
Concrete	--	--	--	+390,213	- 2.08	- 811,643
Earth (on slab)	--	--	--	+130,809	+ 6.50	+ 850,259
Earth pressure	- 23,688	+6.0	- 142,128	- 4,300	- 4.50	+ 19,350
Water load	-181,841	+6.4	-1,163,782	+179,271	+ 5.33	+ 955,514
Backwater	+ 39,476 (net)	+0.5	+ 19,735	+ 1,818	+14.50	+ 26,361
Uplift	--	--	--	-251,075	- 0	+ 0
Total	-166,053	7.98	-1,325,645	+446,736	2.33	+1,039,840

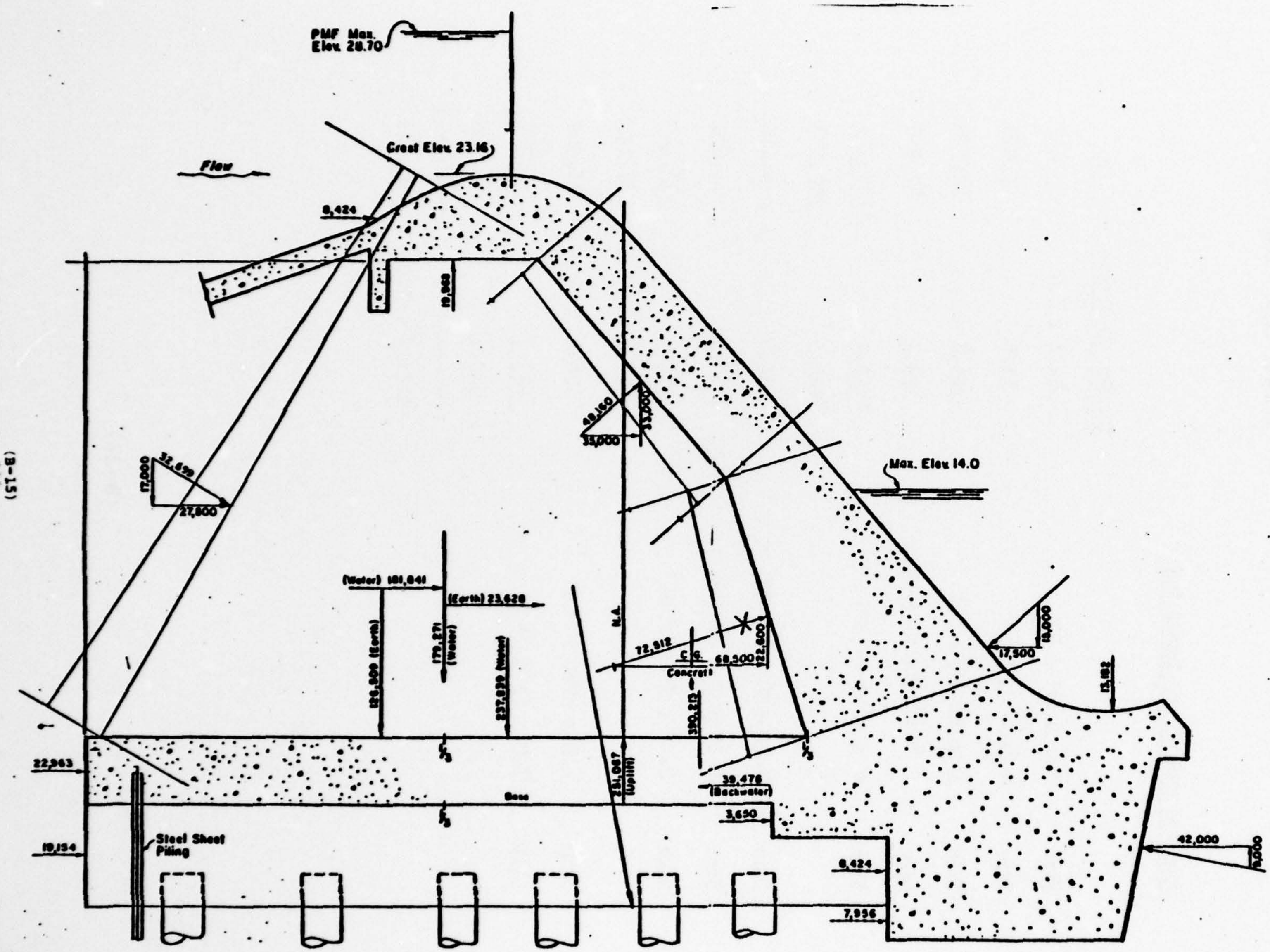
Sliding factor $= \frac{166,053}{446,736} = \underline{\underline{0.37}}$

Stresses on top of 4' wide pier

$$\begin{array}{r} 1,039,840 \\ \underline{-1,325,645} \end{array}$$

$$- \frac{285,805}{446,736} = 0.64 \text{ ft.} = ecc$$

$$\frac{446,736}{4x32.25x144} \left(1 \pm \frac{6x0.64}{32.23}\right) = 24.05 \; (1 \pm .12) = \underline{\left(\begin{array}{l}+26.91 \\ +21.16\end{array}\right.}$$



ORADELL D/
STABILITY GRAP
SPILLWAY SECT

ORADELL DAM

SLUICEWAY SECTION

(10'-2" ctrs) Max. Sec.

Average weight per lineal foot

		Cu.Ft. Concrete	Lbs.
Pier	2' x 6.67' x 23'	306.8 x 150	46,020
Pier	$\frac{1' \times 2'}{2}$ x 23'	23.0	3,450
Pier	3' x 20'-4½ x 5.5'	336.2	50,428
Pier	3' x $\frac{17.5+3.5}{2}$ x 16.5	519.8	77,970
Pier	4' x 15 x 24.375	1462.5	219,375
		2648.3	397,343
Slab	0.83 x 9.0 x 8'-2"	61.0	9,150
Slab	1'-3" x 1' x 8'-2"	10.2	1,530
Slab	1'-4" x 7'-2" x 26'	247.9	37,185
		319.1	47,865
Wall	1.0 x 14½' x 7'-2"	104.0	15,600
Wall	2.0 x 15' x 6'-2"	185.1	27,765
		289.1	43,365
		3256.5 x 150	488,473

ORADELL DAM

SLUICEWAY SECTION

CENTER OF GRAVITY (Gate not inc.)

ONE SLUICEWAY (Near dam)

	Vol. (cu.ft.)	Cu.Ft.	x	Moment	x	Moment
Pier	1 x 2/2 x 23.5	23.5	-0.25	-5.9	23.17	-544.5
Pier	2 x 6.67 x 23.5	313.5	3.34	1047.1	23.17	7,263.8
Pier	3.625 x 3 x 16.33	177.6	8.88	1577.1	33.0	5,860.8
Pier	$\frac{16.33 \times 14.75}{2} \times 3$	361.3	15.17	5480.9	30.34	10,961.8
Pier	18.375 x 6.67 x 3	367.6	15.90	5844.8	21.34	7,844.6
Pier	1 x 2 x 23.5	47.0	1.00	47.0	23.17	1,089.0
Pier	6.67 x 17.5 x 3	350.2	3.34	1169.7	8.25	2,889.2
Pier	18.375 x 18 x 3	992.3	16.00	15,876.8	10.00	9,923.0
		2633.0		31,037.5		45,287.7
Deck	3.375 x 0.83 x 8.16	22.9	2.55	58.4	40.5	927.5
	2.08 x 1 x 8.16	17.0	4.50	76.5	40.5	688.5
	2.0 x 0.83 x 8.16	13.5	8.75	118.1	40.0	540.0
		53.4		231.0		2156.0
Wall	13.75 x 1 x 8.16	112.2	7.17	804.5	34.4	3859.7
Floor	1.25 x 6.67 x 7.16	59.7	3.34	199.4	17.0	1014.9
Floor	1.25 x 18.375 x 7.16	164.5	15.90	2615.6	17.5	2878.8
Wall	2.0 x 16.75 x 7.16	239.9	7.70	1847.2	8.33	1998.4
		576.3		5466.7		9751.8
		3262.7		36,757.2		57,195.5
		x 150	X =	11.27 ft.	Y =	17.53 ft

10.17)489,405

48,122#/ft.
average

ORADELL DAM

SLUICEWAY SECTION Gates Open

Res. W.S. = 28.70, Backwater = 14.0

One Gate Unit = 10'-2" Ctrs. Max. Sec.

Water loads

Upper pier	27.00 x 62.4 = 1685		
	3.03 x 62.4 = 189		Lbs
	2)1874(937 x 24 x 2'	=	44,976
Lower pier	41 x 62.4 = 2558		
	27 x 62.4 = 1685		
	2)4243(2122 x 14 x 4	=	118,832
Floor slab	26.5 x 62.4 x 1.5 x 8.16	=	20,240
Vert. slab	17.0 x 62.4 = 1061		
	3.03 x 62.4 = 189		
	2)1250(625 x 14 x 8.16	=	71,400
Lower x pier	2122 x 13.5 x 6.17	=	176,752

Tailwater

Upper pier	$\frac{6^2}{2}$ x 62.4 x 3	=	3,369
Upper pier	11.5 x 62.4 = 718		
	6 x 62.4 = 374		
	2)1092(546 x 3 x 5.5	=	9,009
Lower pier	28.0 x 62.4 = 1747		
	12.5 x 62.4 + 780		
	2)2527(1264 x 4 x 14	=	70,784
Upper pier	$\frac{2.5^2}{2}$ x 62.4	=	195
Upper pier		=	3,369
Upper pier	6 x 62.4 x 2.5 x 3	=	2,808
Lower X pier	28.0 x 62.4 = 1747		
	12.5 x 62.4 = 780		
	2)2527(1264 x 6.17 x 13.5	=	105,285

Uplift

Approx. balance water load on slabs

Bottom pier =	41 x 62.4 x 4 x 7	=	71,635
	26 x 62.4 x 4 x 16	=	103,834
	33 x 62.4 x 4 x 6.16	=	50,739

Earth Passive Resistance

P = 3.35 x 63 x $\frac{14^2}{2}$ x 10.16 = 210,138# @ 1/3 pt.

(B-18)

ORADELL DAM

SLUICEWAY SECTION

Water

- 44,976	x	22.5	=	1,011,960
- 118,832	x	6.5	=	772,408
- 20,240	x	14.0	=	283,360
- 71,400	x	29.5	=	2,106,300
- 176,752	x	7.0	=	1,237,264
- 432,200		12.52		5,411,292

Tailwater

+ 3,369	x	22.0	=	74,118
+ 9,009	x	17.75	=	159,909
+ 70,784	x	6.25	=	442,400
+ 105,285	x	6.25	=	658,031
+ 195	x	24.50	=	4,778
+ 188,642	x	7.10		1,339,236
+ 3,369	x	22.75	=	76,645
+ 2,808	x	25.6	=	71,885
+ 6,177		24.05		148,530

Uplift

- 71,635	x	3.5	=	250,723
- 103,834	x	14.6	=	1,515,976
- 50,739	x	8.0	=	405,912
- 226,208		9.60		2,172,611

ORADELL DAM

SLUICEWAY SECTION

	Horiz. Load	Horiz. Arm	Horiz. Moment	Vert. Load	Vert. Arm	Vert. Moment
Weight	--			+ 489,405	1.60	+ 783,048
Uplift	--			- 226,208	2.75	- 622,050
Backwater	+ 188,642	9.0	+ 1,697,778	+ 6,117	11.90	- 72,792
Water	- 432,200	12.5	- 5,402,500	--	--	--
Passive earth res.	+ 210,138	4.67	+ 981,344	--	--	--
	- 33,420		- 2,723,378	+ 269,314	0.33	+ 88,206

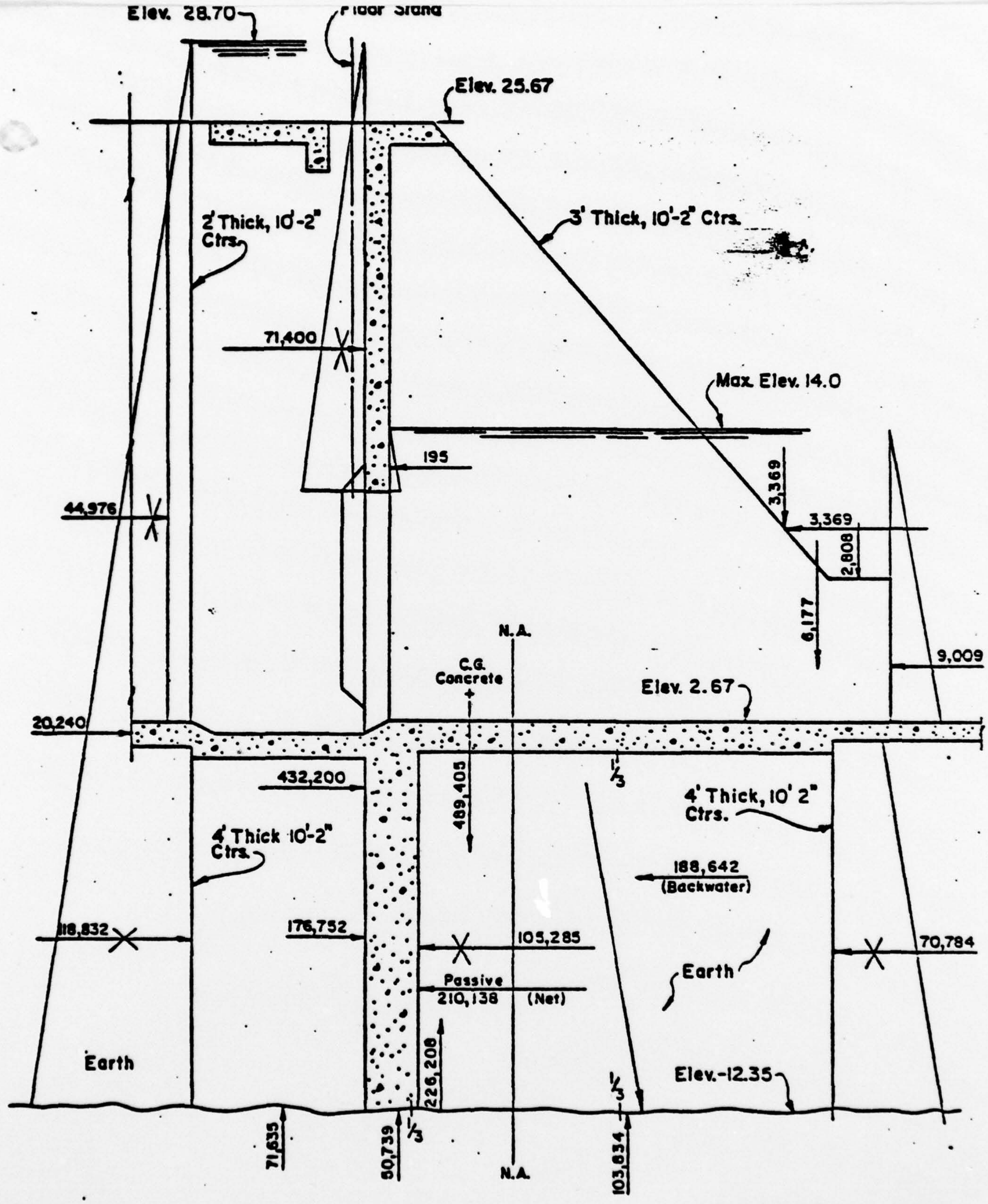
Shear Factor $= \frac{33,420}{269,314} = 0.12$ (Inc. passive resistance)

$$\begin{array}{r} 2,723,378 \\ -\ \underline{88,206} \end{array}$$

$$\frac{2,635,172}{269,314} = 9.78' = \text{ecc.}$$

Stresses

$$\frac{269,314}{4 \times 25 \times 144}\left(1 \pm \frac{6 \times 9.78}{25}\right) = 18.70\ (1 \pm 2.35) = \begin{cases} +62.59 \text{ psi} \\ -25.19 \text{ psi} \end{cases}$$



ORADELL DAM

STABILITY GRAPHICS

SLUICEWAY

Scale: 1" = 5'

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER NJ00258	2. GOVT ACCESSION NO.	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) Phase I Inspection Report National Dam Safety Program Oradell Reservoir Bergen County, N.J		5. TYPE OF REPORT & PERIOD COVERED 9 FINAL rept. 6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) 10 Michael Baker, III, P.E.		8. CONTRACT OR GRANT NUMBER(s) 15 DACW61-78-C-0141
9. PERFORMING ORGANIZATION NAME AND ADDRESS Michael Baker, Jr. Inc. 4301 Dutch Ridge Rd. Box 280 Beaver, Pa. 15009 12 122 p.		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS U.S. Army Engineer District, Philadelphia Custom House, 2d & Chestnut Streets Philadelphia, Pennsylvania 19106		12. REPORT DATE 11 August, 1978 13. NUMBER OF PAGES
14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office)		15. SECURITY CLASS. (of this report) Unclassified 15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

6 National Dam Safety Program. Oradell Reservoir Dam (NJ00258), Hackensack River Basin, Hackensack River, Bergen County, New Jersey. Phase I Inspection Report.

18. SUPPLEMENTARY NOTES

Copies are obtainable from National Technical Information Service, Springfield, Virginia, 22151.

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Dams--N.J.
Oradell Reservoir Dam, N.J.
National Dam Safety Program Phase I
Dam Safety
Dam Inspection

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report cites results of a technical investigation as to the dam's adequacy. The inspection and evaluation of the dam is as prescribed by the National Dam Inspection Act, Public Law 92-367. The technical investigation includes visual inspection, review of available design and construction records, and preliminary structural and hydraulic and hydrologic calculations, as applicable. An assessment of the dam's general condition is included in the report.

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